PRACTICAL HYDRAULICS AND ITS APPLICATION

WITH TABLES, DIAGRAMS AND FULL ILLUSTRATIONS

By

T. B. MADNANI,

L.C.E. (Hons.), Bombay University and A.M.I.E. (former Executive Engineer, P.W.D., NEW DELHI and LAHORE).

VOLUME No. 1
FIRST EDITION

PRINTED BY J. H. HARRIS AT THE TIMES OF INDIA PRESS, BOMBAY (1949).
PUBLISHED BY T. B. MADNANI C/O GRINDLAY'S BANK LTD., FORT, BOMBAY.

PREFACE.

The writer has been employed since 1903 to date on the design and execution of water-supply schemes, sewerage schemes and Hydro-Electric schemes in different parts of India. While working with specialists from England and America, certain notes and Technical details, not available in Books on Hydraulics (generally written by eminent professors with no practical experience) have been collected from time to time.

The specialists kept their trade secrets with themselves and their services were renewed by Government for years, even after they attained the age of 55 years, because they could not be replaced by right men. Though getting princely salaries, they did no research work in this science.

The notes and Technical details collected from different schemes, different books, different Technical Engineering journals, both foreign and Indian, also Encyclopædia Britannica, are now put in the form of this book, which also contains first principles of the different subjects forming the science of Hydraulics as a whole. Numerous examples, fully worked out, are also given for guidance of college students.

The trade secrets of specialists, on which water-supply and sewerage schemes are based, are freely given here to enable young engineers to handle their jobs successfully.

Volume No. 2 of this book, comprising Hydraulic Machines, Pumps, Turbines, Storage Reservoirs and Dams, will soon be given in the Press.

The names of eminent engineers, Indian and foreign, are mentioned along with their achievements here. If any reader wishes to amplify any particular subject, he is welcome to do so and will be remunerated when printing the 2nd Edition.

My acknowledgments are due to all the learned Professors and eminent engineers from whose works, quotations are freely embodied in this book. Their names are also mentioned here. If any reader notices any shortcoming in this book, he is earnestly requested to communicate to the writer.

T. B. MADNANI.

C/o Messrs. Grindlays Bank Ltd.,
Fort, BOMBAY.
1–6–1949.

CONTENTS.

Chapter I.

PRELIMINARY.

Chapter II.

HYDROSTATICS.

Chapter III.

SPECIFIC GRAVITY AND BUOYANCY.

Chapter IV.

GENERAL PRINCIPLES.

CONTENTS—*Contd.*

CHAPTER V.

HYDRO-DYNAMICS.

CHAPTER VI.

DISCHARGE THROUGH SMALL ORIFICES.

CHAPTER VII.

DISCHARGE FROM LARGE ORIFICES AND NOTCHES.

CHAPTER VIII.

WEIRS AND PRACTICAL CASES OF DISCHARGE FROM LARGE ORIFICES AND NOTCHES.

CONTENTS—*Contd.*

CHAPTER IX.

DISCHARGE UNDER VARIABLE HEAD ORIFICES AND SLUICES.

CHAPTER X.

FLOW OF WATER IN PIPES—STEADY MOTION.

CHAPTER XI.

OPEN CHANNELS UNIFORM FLOW.

CONTENTS—*Contd.*

CHAPTER XII.

IRREGULAR OPEN CHANNELS.

CHAPTER XIII.

RIVERS.

CHAPTER XIV.

CANALS, WEIRS AND BARRAGES.

CONTENTS—*Contd.*

CHAPTER XV.

DRAINS AND SEWERS.

CHAPTER XVI.

HYDRAULIC OBSERVATIONS.

CHAPTER XVII.

MISCELLANEOUS.

UNITS AND SYMBOLS EMPLOYED.

Units :—Throughout this work 1 pound, 1 foot and 1 second are taken as the units of weight, length, and time respectively, except where otherwise expressly stated.

Symbols :—The following is a list of the principal symbols employed.

A = Area of a cross section in square feet.

C = Coefficient of discharge.

d = Depth of water in feet.

D = Diameter of pipe in feet.

g = The force or acceleration due to gravity, increase in the velocity of a falling body by 32·17 feet per second in each second or 9·806 metres per second.

H = Maximum head of water in feet.

h = Head of water in feet.

ha = Head in feet required to produce velocity of approach.

L = Length of a notch, weir, pipe, etc. in feet.

u = Coefficient of fluid friction.

P = Pressure at a point in lbs. per square foot.

Ph = Pressure at a depth of h feet in lbs. per square foot.

π = Atmospheric pressure in lbs. per square foot.

= 34 feet high column of water, in a vacuum tube.

= 34 × 62·5 lbs. per sq. foot or 14·696 lbs. per sq. inch.

= 29·92 inches of mercury, in a vacuum tube.

A pressure gauge registers the pressure above atmosphere. Gauge pressure plus the reading of the water-barometer is called Absolute Pressure, positive.

A vacuum gauge registers the water pressure when below atmospheric pressure. The vacuum gauge pressure subtracted from atmospheric pressure gives absolute pressure, negative.

Q = Volume of discharge in cubic feet.

r = Hydraulic mean depth in feet.

s = Area of water surface in square feet.

S = Sine of slope.

t = Time in seconds.

v = Velocity in feet per second.

w = Weight in lbs. of a cubic foot of water.

= 62·4 lbs. (clear) : 62·5 lbs. (with sediment).

x = Afflux in feet.

z = Height of water surface in feet above a datum line.

ω = Angular velocity in radians.

One radian = 57·296°.

Properties of Water.—Pure water consists of two gases, hydrogen and oxygen, in chemical combination in the proportion of 2 volumes of hydrogen to 1 of oxygen, or 1 part of hydrogen by weight to 8 of oxygen. Bulk for bulk, oxygen (sp. gr. 1·102) is 16 times heavier than hydrogen (sp. gr. 0·0689).

Pure water becomes ice at 32° Fah., and steam at 212° Fah. at sea level. It boils at about 1° less for every 520 feet of elevation above sea level for heights within 1 mile.

Water attains its maximum density at 39·2° Fah. It is then about 815 times heavier than atmospheric air. It expands $\frac{1}{12}$ on becoming ice and evaporates at all temperatures.

The compressibility of water under pressure is very slight and it recovers its original volume when the pressure is removed. Under a pressure of one atmosphere (14·75 lbs.), a column of water 1 foot high would be compressed about $\frac{1}{1800}$ of an inch.

Water has a bulk elastic modulus when under compression and will store up energy in the same manner as a solid. The value of the bulk elastic modulus of water under compression is 300,000 lbs. per square inch.

Water will withstand a slight amount of tension owing to the molecular attraction between the parties, which will cause an apparent shear resistance between adjacent two layers ; this phenomenon is known as viscosity.

The ratio between the density of any liquid and the density of water is known as the specific gravity of that liquid.

No liquid can exist as a liquid at zero pressure ; in fact all known liquids vaporize at various pressures above zero, depending on the temperature.

Water vaporizes at pressure of 0·34 lb. per sq. in. at 20° C., below this pressure it can not exist as a liquid. Care must be taken to prevent the pressure of water getting below 8 feet of water absolute, at which pressure the dissolved gases are given off and vaporization is also liable to commence.

WEIGHTS AND MEASURES.

Water 1 C. Ft. = 62·41 lbs. (clear) = 62·5 lbs. (with sediment).
= 64 lbs. (sea water) = 6¼ gallons.
An Imperial gallon of water = ·16 cubic foot.
= 10 lbs. = 8 pints = 70,000 grains.
One United States gallon = 5/6 of an Imperial gallon.
One metre = 3·2809 feet. = 39·37 inches.
= 100 centimetres.
One cubic metre = 35·317 cubic feet.
= 1000 Kilograms.
One kilogram = 2·2055 pounds, avoirdupois = 1000 gms.
One litre = 61·027 cubic inches.
= ·2201 Imperial gallon.
= 1000 c.c.
1 lb. = 453·6 grammes.
= 16 oz. = 7000 grains.
1 inch = 2·51 centimetres.
One French inch = ·02707 metre.
= ·0888 English foot.
Bulk elastic modulus of water = 300,000 lbs. per square inch.
One knot = 6080 feet per hour = one nautical mile per hour.
Common Logarithms × 2·303 = Hyperbolic or Naperian Logarithm (to Base e)
Volts × amperes = Watts
One Horse-Power = 33,000 Ft.-Pounds per minute.
= 746 watts.

CHAPTER I.

PRELIMINARY

1. Introduction

(*a*) The term fluid is applied to

(*i*) Liquids like water, milk, alcohol which have definite volume and are incompressible but they take the form of the vessel in which they are poured.

(*ii*) Gases like air which have no form and are compressible to an indefinite extent.

(*b*) A fluid is said to be viscuous like honey, when it offers some resistance, though small, to change of form. In this book fluid means ordinary water only.

(*c*) Hydromechanics is that science which deals with,

(*i*) Laws regulating the equilibrium of water at rest.

(*ii*) With the mathematical theory of water in motion.

No. (*i*) is called the science of Hydrostatics and No. (*ii*) is called the science of Hydrodynamics.

Some of the principles of No. (*i*) and No. (*ii*) along with results of practical observations, are grouped together, so far applicable to engineering practice, into a science called 'Hydraulics' which deals with the study of water pressure, buoyancy, the flow of water and hydraulic machinery, such as pumps, turbines, and accumulators.

(*d*) History and Remarks :—

The origin of the hydraulic arts is lost in antiquity, but probably the oldest branch of the applied arts is to be found in the irrigation schemes carried out in Mesopotamia, Egypt and China, several thousand years ago. The lake Moesis which was constructed about 4,000 years ago to hold and store the flood waters of the Nile, was much larger than any modern reservoir.

In Greece and Rome, the remains of great aqueducts can be traced and the people of these countries knew all about the size and right grade of the acqueducts.

In fifteenth century, the great Italian Leonardo da Vinci wrote a book entitled " on the motion and measurement of water ", dealing with flow of water over weirs and through orifices.

In seventeenth century Galileo* deduced the fundamental Law $V = \sqrt{2gh}$. His pupil Torricelli discovered that 'Discharge of water through an orifice is proportional to the square root of the head of water on the orifice.'

In 1775, the well-known Chezy formula was discovered. $V = C\sqrt{rs}$ for flow in conduits. About the same time Daniel and John Bernoulli established their famous theorem, known as the Bernoulli† Theorem, on which so many hydraulic principles are based.

* Galileo :—(1564-1642). The great Italian Astronomer of Pusa. While a youth discovered the Law of Pendulum vibration by seeing a lamp swinging from the roof of a cathedral in Pusa. He constructed the first telescope and made many astronomical studies. He published a treatise on Hydraulics in 1612.

† Daniel Bernoulli :—(1700-1782). Native of Basil, a town on both sides of River Rhine in Switzerland, where his distinguished family members were mathematicians of high order. He was professor of Mathematics for some years at St. Petersburg. He published his Hydrodynamica (1738). He established his " Bernoulli's Theorem ".

During the nineteenth century, progress in hydraulic development was rapid and during twentieth century much attention has been paid to hydraulic observations. The number of experimenters is so large that it is possible only to mention the more important. They are as under :—

Castelli, Torricelli, Pascal, Mariotte, Newton, Pitot, Bernoulli, D'Alembert, Dubaut, Bossut, Prony, Eytelwein, Mallet, Vici, Hachette, Bidone, Michelotti, D'Aubuisson, Castel, and Borda :

For flow in pipes, Weisbach, Coulomb, Venturi, Couplet, Darcy, Lampe, Hazen, Poiseuille, Reynolds, Smith, Stearns.

For flow through apertures, Poncelet, Lesbros, Weisbach, Rennie, Blackwell, Boileau, Ellis, Bornemann, Thomson, Francis, Unwin, Fteley, Stearns, Herschel, Steckel, Fanning and Smith.

For flow of turbid water in canals and earthen channels, Mr. R. G. Kennedy, Chief Engineer, P. W. D., Irrigation Department, Punjab, India (1900).

Mr. Gerald Lacey, Superintending Engineer, P. W. D., United Provinces of Agra and Oudh (India), who has devoted much of his precious time to the study of this subject, and evolved very interesting new theories and formulæ (1948).

For large canals, like Ganges canal, Col. Cunningham.

For foundations of Weirs in sandy beds of rivers in Northern-India, eminent Indian Engineers, Khosla and Kanwarsain (1948).

For canals in Sind, Mr. Inglis.

For open channels carrying more or less clear water, Darcy and Bazin.

On Mississipi River, Humphreys and Abbott.

For series of Coefficients for mean velocity, Swiss Engineers, Ganguillet and Kutter.

For Tables of coefficients, Jackson.

On rod floats, Francis.

On current meters, Stearns.

For inundation canals in Sindh and south Punjab, Muslim Engineers, probably imported from Egypt centuries before the British conquered Sindh and Punjab (1843-1850).

CHAPTER II.

HYDROSTATICS

2. Laws, Pressures, Etc.

(*a*) Hydrostatic Laws.

(*i*) Water, an almost incompressible liquid, weighs nearly one thousand oz. or $62\frac{1}{2}$ lbs. per cubic foot. A gallon of water weighs 10 pounds. Water becomes solid in the state of ice at 32° F, and gaseous in the form of steam at 212° F. Water has, when under compression, bulk elastic modulus 300,000 lbs. per sq. inch.

(*ii*) Pressure of water on any plane surface in any position.

This is equal to the weight of a column of water whose base is the area of surface and whose height is the depth of the centre of gravity of that surface below the free surface level of the water.

(*iii*) The direction of the pressure on a surface is always perpendicular to that surface, whatever be its position.

(*iv*) The resultant pressure of water on a body immersed or partly immersed in it, is equal to the weight of volume of water displaced and acts vertically upwards through the centre of gravity of this volume. This centre of gravity is called the " centre of buoyancy ".

If the body floats, it follows that the weight of water displaced is equal to the weight of the body.

(*v*) *Pressure at a point*:—The pressure at a point is the pressure per unit of area. If the unit is one square foot and the depth of a point below the free surface of water is h feet, the pressure Ph = (1 sq. foot $\times$ h feet) C. Ft. $\times 62\frac{1}{2}$ lbs., or if we denote the weight of a cubic foot of water namely 62.5 lbs. by W.

P = Wh. . . . . (I). The pressures at any two or more points at the same level in a liquid are evidently equal.

(*vi*) The difference between pressures at two points in a mass of liquid at rest, depends only upon the vertical distance between the horizontal planes through those points.

(*vii*) If the pressure at any point in a mass of liquid be increased, then the pressures at all other points are increased by the same amount.

(*viii*) In a mass of liquid at rest, surfaces of constant pressure are horizontal planes.

(*ix*) Average pressure over a plane surface immersed in a mass of liquid at rest is equal to the pressure at the centre of gravity of the surface.

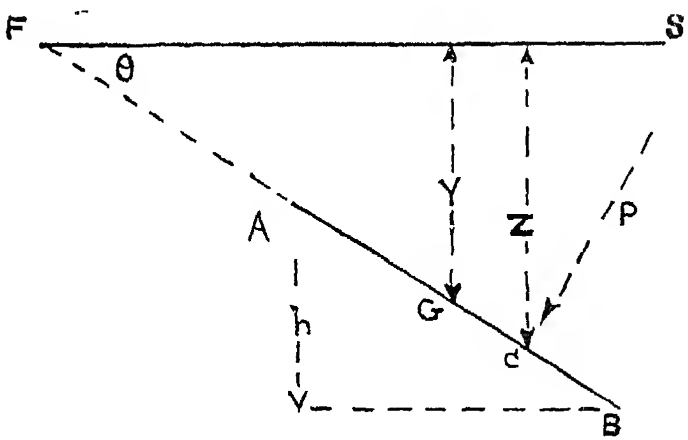
The centre of pressure on such a surface, if the surface is horizontal, coincides with its centre of gravity. If the surface is vertical or sloping, the centre of pressure is always below the centre of gravity of the surface and is found by considering that the pressure is an uniformally varying stress, whose intensity at a given point varies as the distance of that point from the line where the given plane surface (produced if necessary) intersects the upper surface of the liquid (See page 165 Rankine).

(*b*) Pressure on surfaces :—

I :—Rectangular plane surface, submerged and inclined :—

AB = a plane plate, 1 foot wide inclined at an angle θ with the Horizontal.

Fig. 1



FS = free surface of water.

A = area of plane surface AB in square feet.

G = Centre of gravity of the plane at a depth Y feet below the free surface.

C = Centre of total pressure P in pounds,
On AB at a depth Z feet.

The pressure P will act at right-angles to line AB or normal to surface AB.

I = The moment of Inertia in foot pounds of the plane AB about a horizontal axis through its centre of gravity G = AK^2.

K = Radius of Gyration about Horizontal axis through centre of gravity $= \sqrt{\left(\frac{I}{A}\right)}$

$$\text{Here } K^2 = \frac{1}{12}h^2$$

$$\text{Then } P = WYA$$

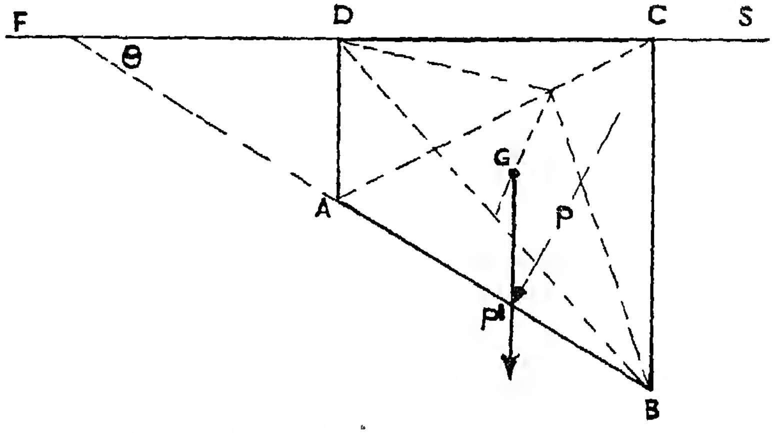
$$Z = Y + \frac{I \sin \theta^2}{AY}$$

$$= Y + \frac{K^2 \sin \theta^2}{Y}$$

Rectangular plane surface submerged and inclined :—

Graphic Diagram.

Fig. 2



AB = a piece of plate submerged, length AB, width 1 foot, *inclined* at angle θ with the horizontal line Fs which is free surface of water at rest :

The known lengths AD, AB, BC and the angle θ to be plotted on a suitable scale.

Pressure on plate AB = weight of mass of water ABCD = area of ABCD in sq. feet $\times 1 \times$ 62.5 Lbs. ; the depths AD and BC are known : Length AB is known

DC = AB cos θ :

Thus total resultant pressure $P = \left(\frac{AD+BC}{2}\right) \times AB \cos \theta \times 1 \times 62.5$ lbs. A vertical line through G, the centre of Gravity of mass of water ABCD, cuts AB at point P^1 : at this point the pressure acts along line PP^1, normal to surface AB : P^1 is the middle point of 'the width of plate AB at P^1' and not of the length AB :

II :—Circular surface submerged and inclined :—

d = Diameter of circle

c = Centre of circle = Centre of gravity

Y = Depth in feet of the centre below the horizontal and free surface of water.

θ = The angle of inclination with the Horizontal surface say 30°

Total Pressure $P = 62.5 \times \frac{\pi d^2}{4} \times Y$ lbs.

K = Radius of Gyration

$K2 = \frac{1}{16} d^2$ in case of a circle

$$Z = Y + \frac{K^2 \sin \theta^2}{Y}$$

$$= Y + \frac{d^2 \sin \theta^2}{16Y}$$

$$= Y + \frac{d^2}{64Y}, \text{ if } \theta = 30°$$

III :—Any vertical surface :—

In this case $\theta = 90°$

And $P = WAY$

$$Z = Y + \frac{K^2}{Y}$$

IV :—Case of a vertical wall : Length one foot :—

Holding h feet of water ; Top of wall at F. S. level.

Y or depth of centre of gravity of wall $= \frac{h}{2}$

K^2 for a rectangle of height h is $\frac{h^2}{12}$

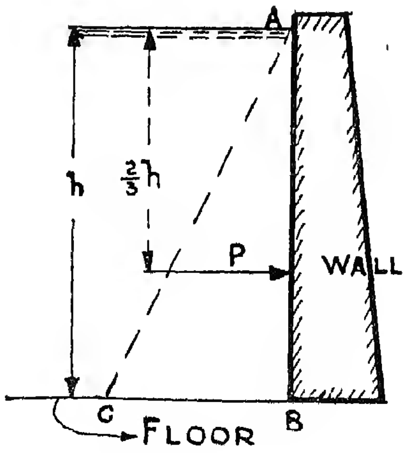
Thus $P = WYA = \frac{Wh}{2} h = \frac{Wh^2}{2}$, per foot run of wall.

Z = Depth below F. S. Level of centre of pressure $= Y + \frac{K^2}{Y} = \frac{h}{2} + \left(\frac{h^2}{12} \times \frac{2}{h}\right) = \frac{2}{3} h$

Fig. 3

Wall A B with a vertical face :—

Graphic method



Vertical wall length 1 foot, holding up h feet of water.

Pressure at B = Wh acting horizontally.

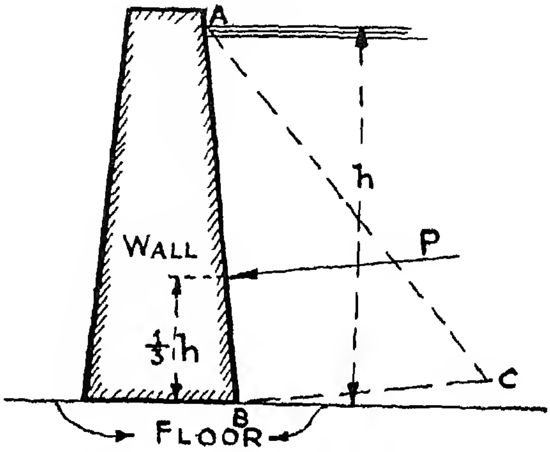
Plot to some scale, a triangle, AB = BC = h

P = the total resultant pressure

$= \text{area of triangle ABC} \times 1 \text{ foot} \times W = \frac{Wh^2}{2}$ per foot run of wall.

The pressure acts horizontally, through the centre of gravity of the triangle A B C at a point $\frac{2}{3}$ h below A, the free surface of water.

Fig. 4



V :—Wall AB with sloping face :—

Pressure at B = wh lbs.

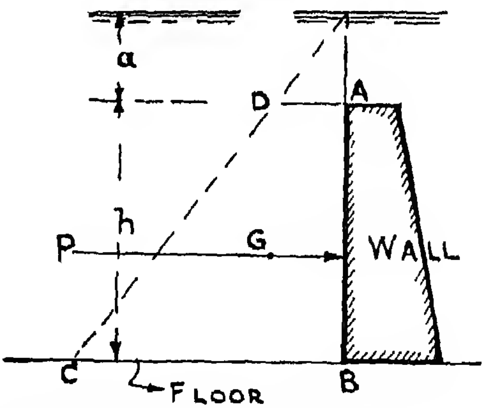
Plot the triangle A B C to some scale, with BC = h feet and normal to AB.

Total resultant pressure per foot length of wall = area of triangle ABC × 1 × 62·5 lbs.

$= \frac{1}{2}$ (AB×BC) × 1 × 62·5

This total pressure acts normal to the surface AB, through the centre of gravity of the triangle ABC at a point $\frac{2}{3}$ h below the free surface level of water.

Fig. 5



VI :—Vertical wall submerged, width 1 foot.

Pressure at point A = wa

Pressure at B = (h + a) w.

Show these pressures by DA (= a feet) and BC (= h + a) and complete the diagram ABCD.

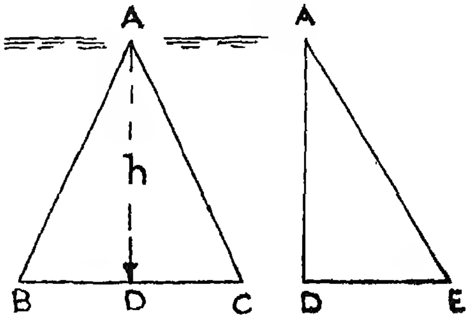
P = Total resultant pressure on AB, per foot length of the wall AB = area of Fig. A B C D × 1 × 62.5 lbs., acting horizontally through the centre of gravity G of Fig. A B C D.

$$P = Wh\left(a + \frac{h}{2}\right) \times 1 \times 62{\cdot}5$$

Centre of pressure on the line A B, from the free surface of water $= \frac{2}{3} \quad \frac{3a^2 + 3ah + h^2}{2a+h}$

VII :—Triangle immersed vertically in water with apex at surface.

Fig. 6



$$P = \frac{BC \times h}{2} \times \frac{2}{3}\ hw$$

$$= \frac{1}{3} wbh^2 \text{ where } BC = b$$

Pressure along the line AD is represented by the area of the triangle ADE, DE being $= h$

Pressure along line BC $=$ area of rectangle BC $\times$ h.

Pressure on the triangle ABC $=$ weight of mass of water contained in a pyramid with base (BC $\times$ h) and apex at A and 4 triangular sides $= \frac{1}{3}$ (BC $\times$ h $\times$ h) W $= \frac{1}{3}$ Wbh^2.

Centre of pressure $= \frac{1}{4}$ h from BC, on line AD.

VIII :—Triangle, base at surface.

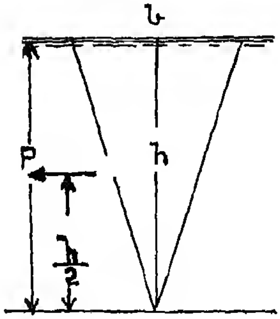
Graphic method.

$$P = \frac{bh}{2} \times \frac{h}{3} W = \frac{1}{6} Wbh^2$$

P $=$ Weight of mass of water contained in a pyramid with triangular base $\frac{bh}{2}$ and height h.

$$P = \frac{1}{3}\left(\quad \frac{h}{2} h\right) W = \frac{1}{6} Wbh^2$$

Fig. 7

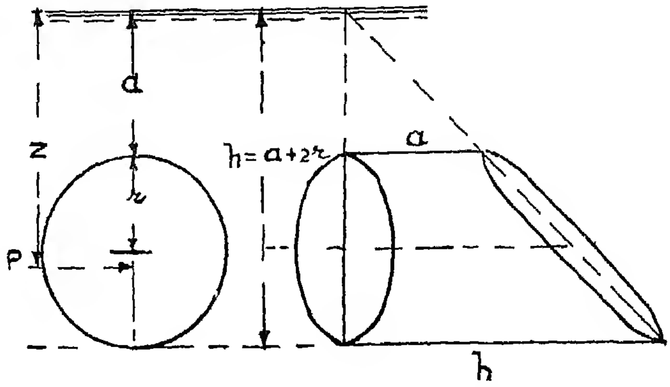


Depth of centre of pressure from surface of water $= \frac{h}{2}$

IX :—Circle, top-edge submerged.

Centre of gravity is the centre of the circle.

Fig. 8



$P = W\pi r^2 (a+r).$

Graphic method.

Pressure on circle = weight of column of water $\left(\text{Frustrum of a cylinder with base } \pi r^2 \text{ and height } \frac{a+h}{2}\right)$
$= W\pi r^2 (a+r).$

Depth of centre of gravity of this frustrum lies on the axis of the cylinder and is $= a+r$ from water-surface.

Z = Depth of total resultant pressure from water surface

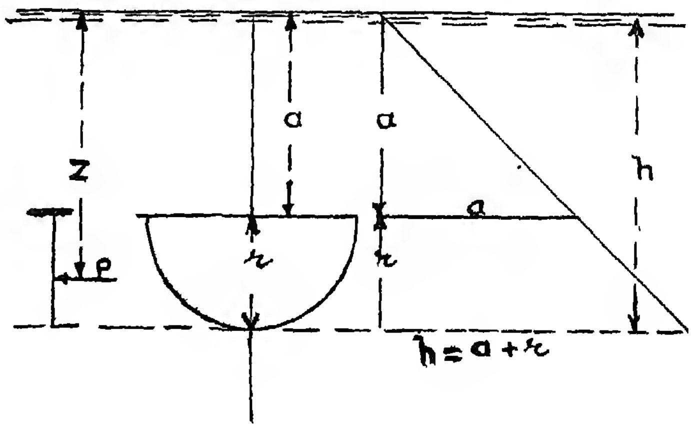
$$= a + r + \frac{r^2}{4(a+r)}$$

X :—Circle top edge at surface level.

Here $a = 0 \therefore P = W\pi r^3 : Z = 1{\cdot}25r.$

XI :—Semi-circle, Diameter submerged.

Fig. 9



$$P = \frac{\pi r^2}{2}(a + .42r)$$

Graphic method.

P = Pressure on semi-circle = weight of a column of water (= Frustrum of semi-cylinder with base $\frac{\pi r^2}{2}$ and height $\frac{a+h}{2}$

$$= \frac{Wr^2}{2}\left(\pi a + \frac{4}{3} r\right).$$

Depth of centre of gravity from water surface

$= a + {\cdot}42\,r$

Z = Depth of centre of pressure from water surface

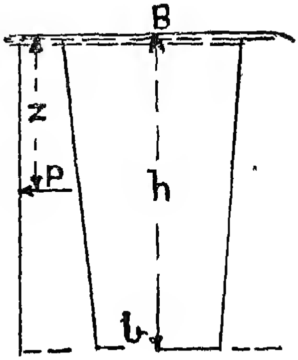
$$= \frac{3\pi(4a^2 + r^2) + 32\,ar}{4(3\pi a + 4r)}$$

XII :—Semi-circle, Diameter at surface level.

Now $a = 0 \quad P = \frac{2}{3} Wr^3$

$$Z = \frac{3}{16} \pi r$$

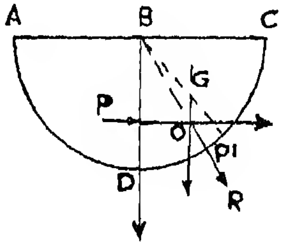
Fig. 10



XIII :—Trapezium, top edge at surface of water.

$P = \frac{1}{6} wh^2 (B+2b)$. Distance of centre of gravity from surface of water $= \frac{h}{3} \quad \frac{B+2b}{B+b}$. Depth of centre of pressure from surface of water $= \frac{h}{2} \quad \frac{B+3b}{B+2b}$

Fig. 11



XIV :—Channel of semi-circular cross section.

Radius say 3 feet ;

Length of channel say 1 ft.

,, closed at two ends by 2 discs ADC, semi-circular in shape, pressures on which being equal and opposite, are neglected here :

Cross section ABCD

Pressure on curved surface ADC = weight of mass of water ABCD acting along the line BD, at centre of pressure P :

Consider the pressure on curved part DC only :

The liquid mass BDC is in equilibrium, due to the following sets of forces :—

SET (*a*) The forces along the plane BD due to pressure of water on both sides, neutralise each other : we consider a set of forces acting along BD, towards the mass of water BDC only :

SET (*b*) The weight of mass of water BDC acts vertically through the centre of gravity, point G, situated on line BG which bisects the angle DBC.

SET (*c*) Liquid pressures on curved surface DC = reaction from the same surface on the mass of water BDC.

Thus the mass of water BDC is kept in equilibrium by three sets of forces.

SET (*a*) = Resultant force on BD = area of surface BD$\times$1 (length of channel) $\times$ 1·5 feet (depth of centre of gravity of BD below the water surface) $\times 62{\cdot}5 = 3\times1{\cdot}5\times62{\cdot}5 = 281{\cdot}25$lb; and this acts through the centre of pressure, point P on line BD, such that BP = 2 PD :

SET (*b*) = the weight of mass of liquid BDC = $\frac{1}{4} \times \pi \times 3 \times 3 \times 1 \times 62 \cdot 5$ lbs. = 441·6 lbs.

This acts through pt. G, the centre of gravity of mass BDC such that BD = $\frac{2}{3} \times 3 \times \frac{2\sqrt{2}}{\pi} = 1.8$ feet.

SET (*c*) The pressure (*a*) and (*b*) intersect at point O, through which their resultant must pass and this resultant is the pressure on the curved surface DC and normal to it; therefore along the line BOP′, the point P′ being the centre of pressure on the surface DP′C.

This resultant pressure = R

$$= \sqrt{\left\{ (281 \cdot 25)^2 + (441 \cdot 6)^2 \right\}} = 535 \cdot 7 \text{ lbs.}$$

XIV :—Pressure on base of a cylinder, full of water, standing vertically with base horizontal.

Total pressure = Total weight of water acting vertically through the centre of gravity of the mass of water, centre of pressure being the centre of base.

In case of cone, total pressure on base = area of the base of the cone (square feet) × height of the cone (feet) × 62. 5 lbs. This is exactly the same as in the case of a cylinder, standing vertically, which holds water 3 times the amount, contained in a cone of the same height.

This anomaly is explained by the fact that a cone has slanting sides and the water pressures acting normal to the sides have their vertical components deflected vertically towards the horizontal base.

Pressure 120 lbs. per sq. foot, lifting up from inside hemispherical dome of 2 ft. radius, results in total lifting force, equal to projected area normal to the given direction (πr^2) × intensity of pressure 120 lbs. = 1509 lbs.

3. ELEMENTARY HYDROSTATIC MACHINES.

i. Hydraulic Press.

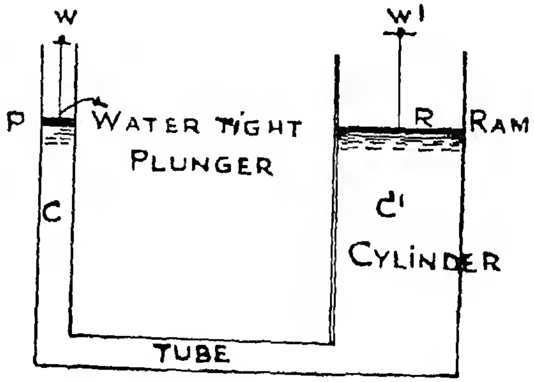
Rule *vii* in article 2 is made use of in the construction of a machine called Hydraulic Press, an instrument, the working of which illustrates the principle of the equal transmissibility of fluid pressure.

A = area of plunger P say one □″

B = area of Ram R say 10 □″

W = weight placed on plunger P say one lb.

Fig. 12



To keep the two, P & R in horizontal position weight W′ is placed on R.

Pressure exerted on R from below = 10 × w

∴ W′ = 10 W or W : W′ : : I : 10

: : A : B

This is the relation between the weights, when the instrument rests in equilibrium with P and R in the same horizontal place.

Thus with a force of 1 lb. (w), a weight of 10 lbs. (w′) can be lifted. The mechanical advantage is $\frac{W'}{W} = 10 = \frac{\text{Area of R}}{\text{Area of P}}$

The Hydraulic lifting Jack is also constructed on this principle.

ii. Atmospheric Pressure :—Surrounding all things on the earth is atmosphere, whose weight acts on all things equally in all directions, with an intensity of 14·7 lbs. per square inch.

In many questions relating to engineering, the pressure of the atmosphere may be left out of consideration, as it acts with sensibly equal intensity on all sides of the body exposed to it and so balances its own action.

To measure the pressure of atmosphere, an instrument has been made, called " Barometer ". It is a glass tube with a glass cup at bottom, full of mercury. The top end is closed hermetically and the bottom end dips in mercury cup. The height of the column of mercury in the tube indicates the atmospheric pressure and is 30 inches at the ground surface. As we go up in the air, the atmospheric pressure diminishes because the density decreases and the height of the mercury column also diminishes. This furnishes a means of estimating the height ascended, and finding out the levels of tops of Mountains and Hills.

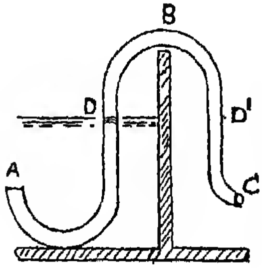
A Rough Formula is

$H = 60{,}000\,(\text{Log } R - \text{Log } r)$ . . . . Where H is the height in feet, R and r the readings in inches at the lower and upper stations respectively.

Since mercury is $\underline{13 \cdot 6}$ times as heavy as water, the height of the water column which can be supported by atmospheric pressure is $\underline{13 \cdot 6} \times 30 \times \frac{1}{12}$ or 34 feet.

The pressure of water is measured by some type of gauge, which registers the pressure above atmosphere. This is called gauge pressure. Gauge pressure + reading of water barometer (34 feet generally) means absolute pressure. If the pressure of water is below atmospheric pressure, it is measured by a vacuum gauge, which gives the amount, the pressure is below the atmosphere ; this must be deducted from the atmospheric pressure in order to obtain absolute pressure. Thus if the reading of the vacuum gauge is 24 feet of water, the absolute pressure is 34 — 24 = 10 feet of water.

Fig. 12A



iii. Syphon :—Let a bent tube ABC (fig. 12A) be filled with water, and have its ends closed. Let one leg AB be placed in a vessel of water, and let the ends A & C be then opened. Water will flow out through C until the level of the water surface in the vessel falls to C or A whichever is higher.

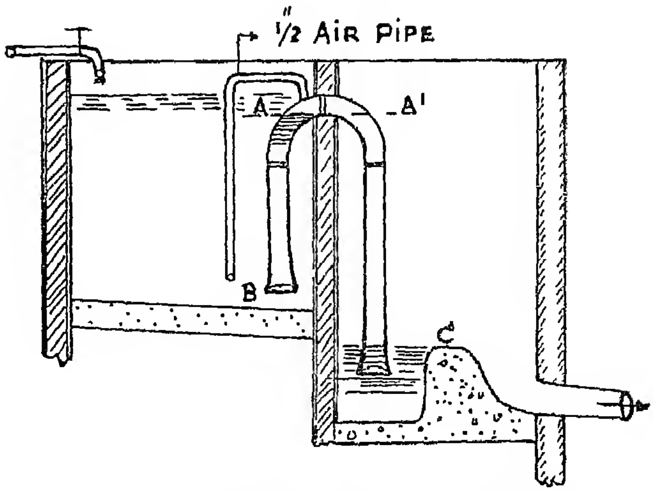
The liquid in the tube being continuous, the pressures at any two points in it at the same level are equal. Thus the pressures at D and D^1 must each be π. But this is the pressure at C, so that the column of water CD is unsupported, and must fall out. The rest of the water in the tube must follow it ; for, if there were a break of continuity, a vacuum would be formed, which is obviously impossible unless the point B rise 34 ft. above the water surface at D. The pressure in the portion DBD^1 of the tube is less than π, so that if a hole were made in this portion the air would rush in, the water would fall in both legs, and the syphon could not act.

In the sketch are two tubes, joined at top by two bends with air-tight joints. This forms a syphon.

The inner leg is placed in an empty tank, lower end being about 3 inches above the floor. The outer leg is placed in a tank with a sump full of water; at bottom, the outer leg dips into the sump by about 4 to 6 inches. This forms a "seal" at C.

There is an air pipe ½ inch diameter, fixed on to the inner leg as *shown in the sketch, its* lower end being ½ inch above the lower end of the inner leg.

Fig. 13



When the water tap is opened, water rises in the tank and also in the inner leg and in the air pipe. The atmospheric air inside the tube is compressed and escapes through the seal at C: when the water level reaches the line AA^1, the water trickles down the outer tube and passes out of the seal C and creates Torriceli's* vacuum in it.

The Atmospheric pressure on the surface of water in the tank A forces the water up the inner tube, to fill up the vacuum in the outer tube; thus a continuous column of water is formed in the tube.

Since the end at C is at a lower level, the water rushes out at C, and the water in the tank continues to rush up the tube and flow at C. The tank is emptied. If the tap is open, the operation of filling the tank A and emptying it by the tube goes on continuously.

The *force,* with which the water comes out at C, is a measure of the quantity of water, and depends upon the difference of level between points B & C.

When the water from the tank A is syphoned out and the tank is empty, the atmospheric air rushes into the syphon at the point B to fill up the vacuum created inside the syphon when the water flows out.

Often a little vacuum is left inside the syphon. When the water rises in the tank and in the inner leg AB, the partial vacuum in the syphon, before the water level reaches the line AB, helps the syphon to work partially. The water trickles out at the point C and goes to waste. The water level in the tank A never reaches the line AB and the tank is not syphoned out at once, we then say the syphon leaks.

To prevent this, an air pipe say ½ inch diameter is fixed on the top of the syphon as shown in the sketch; the lower end of this pipe should not dip into the water left at the bottom of the tank but should be open and free to atmosphere and slightly above the water surface in the bottom of the tank. When the syphon gets empty, the air rushes into the inner leg through this air pipe and destroys partial vacuum.

The water-supply tap should be fitted with a reverse ball-valve to enable more water to come out of the tap, when the syphon starts to work, to accelerate the creation of the Torricelli's vacuum in the outerleg.

* Torriceli:—Evangelist (1668-47), Galileo's pupil. He invented the barometer.

The writer had considerable experience of putting up flushing tanks with syphons from 6 inches to 15 inches diameter at suitable points on sewers in New Delhi, while employed as a public-health engineer (1913-1926).

There are several kinds of flush-tanks in the market, the makers claiming special merits for their make, but the principle involved is that of the syphon.

Examples · (1) A hemispherical dome of 3 feet radius contains water under a pressure of 120 lbs. per sq. ft. Find the total force tending to lift the dome.

The horizontal projected area of the semi-dome $= 9\pi$ sq. feet. The total vertical lifting force $= 120 \times 9\pi = 120 \times 9 \times 3\cdot14 = 3391$ lbs.

(2) A hydraulic press has a ram of 6 in. diameter and a plunger of 1 inch diameter.

(*i*) What force will be required on the plunger to raise a weight of one ton on the ram ?

(*ii*) If the plunger had a stroke of 10 inches how many strokes are required to lift the weight 3 ft. ?

(*iii*) If the time taken to lift the weight is 12 minutes, what horse power would be required to drive the plunger. All losses due to friction are neglected and the motion of the weight is continuous.

(*i*) $$\frac{\text{F the force on plunger}}{\text{weight on the ram (2240 lbs.)}} = \frac{\text{Area of the plunger}}{\text{area of the ram.}} = \frac{1}{36}$$

$$\therefore \quad F = \frac{1}{36} \times 2240 = 62\cdot22 \text{ lbs.}$$

(*ii*) As the work done by plunger in n strokes equals work done by the ram

$$n \times \frac{10}{12} \times 62\cdot22 = 2240 \times 3 \text{ foot pounds}$$

$$n = 130 \text{ number of strokes.}$$

(*iii*) Horse Power required $$\frac{130 \times \frac{10}{12} \times 62\cdot22}{\text{(minutes) } 12 \times 33000}$$

$$= \cdot017$$

(3) In a hydraulic press, the ram is 10 inches and the pump plunger 1 inch diameter, the leverage for working the pump 16 to 1. What is the velocity ratio of the pump handle and ram ? Actually a force of 30 lbs. exerts a pressure of 44,000 lbs. on the press table. What is the efficiency ?

Answer : 1,600 ; 91·7 per cent.

CHAPTER III.

Specific gravity & buoyancy.

4. *Specific gravity* :—The specific gravity of a substance is the ratio of the weight of any volume of the substance to the weight of equal volume of water. Thus the specific gravity of mercury is 13·6, that of water being 1·00.

Knowing the specific gravity of a substance, the weight of any given volume of it can at once be found.

Water vaporizes at a pressure of 0·34 lbs. per square inch at 20°C. Below this pressure it can not exist as a liquid. When the pressure of water gets below 8 feet of water absolute, the gases dissolved are given off and the vaporisation of water begins.

5. *Flotation* :—It is obvious from last article that a body will float or sink in water according as its specific gravity is less or greater than unity (specific gravity of water).

6. *Buoyancy* :—(*i*) *Body immersed in water*.—A body immersed in water loses, in effect its weight by the weight of the volume of water displaced by that body. The downward force on the body is due to gravity while the upward force is the resultant of upward pressures of the water in which the body is floating. This resultant is equal to the weight of the water displaced by the body and is known as force of buoyancy.

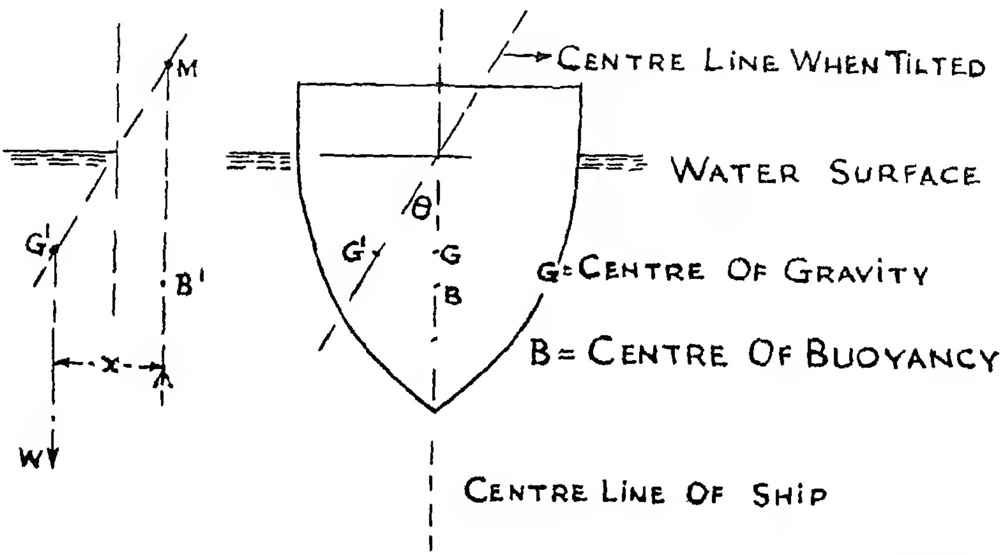
To keep the body afloat, the centre of gravity of the body and the centre of gravity of the volume of water displaced must lie on the same vertical line, the force of buoyancy being equal to the weight of the body. This is known as Archimedes principle*.

For stability of ships, see Sir William White's naval Architecture.

(*ii*) *Conditions of equilibrium of a floating body* :—If the floating body is given a slight angular displacement, such as the rolling of a ship, after which it returns to its original position, the body is said to be stable.

If on being given a slight displacement, it heels farther over, it is said to be in unstable condition. But if the body is given a slight displacement into a new position and it remains at rest in that new position, the body is then said to be in neutral equilibrium.

Fig. 13A



For stable condition G & B must lie on the same vertical line. Suppose the ship tilts over a little on one side, the centre line also tilting through the angle θ. The centre of gravity moves towards the point G^1, the centre of buoyancy moves in opposite direction towards the point B^1.

Thus there are two forces, acting on the ship, one its weight W acting vertically through G^1, trying to bring the ship to its normal position; the other force of buoyancy acting vertically upwards through B' and equal in magnitude to W, trying to bring the ship to its original position. The two forces form a couple called righting couple, the moment being WX, X being the distance between G^1 & B^1.

A vertical line through B^1, cuts the centre of the ship when in tilted position at point M, called Meta-centre, so defined by Bougier in 1746. The distance MG is known as *Meta Centric height*, θ being assumed very small (Traite du Navire by Bougier).

Thus the ship behaves like a Pendulum, suspended from point M, the point G acting as a bob of the Pendulum. The ship thus rotates round the point M, called by some as the instantaneous centre of rotation; M can be nearly a fixed point, if the angle of tilting is very small.

Ships are so designed that the point M is generally at the water line.

* Archimedes 287-212 B.C. A Greek Geometrician and Philosopher of remarkable power.

If M is above G, the ship is stable. If M is below G, the ship is unstable. If M coincides with G, the ship is in neutral position.

In the interests of the safety of tho ship, tho point M should be above G, the distance varying from 1 ft. to 4 ft. with the size and shape of the vessel.

The boats carrying cargoes along the sea-coasts of India are so designed as to keep the centre of gravity below the water-level when loaded. Officers in charge of modern steamers sometimes put ballast in the bottom part of the ship during times of insuffieient cargo to keep tho point G below water line and thus make the ship stable.

The ancients knew the art of building wooden ships and large boats so well that their boats carried cargo and passengers from one part of the World to another across deep and rough Oceans.

Examples :

1.—A battleship weighs 13,000 tons. On filling the ships boats on one side with water (this weighing 60 tons and its mean distance from the centre of the boat being 30 feet) the angle of displacement of a plumb line is $2^\circ\ 16'$ ($\tan\theta = \cdot 0396$). Determine the meta centric height for rolling displacements. *Answer :* 3 ft. 6 inches.

CHAPTER IV

GENERAL PRINCIPLES

7. Definitions

(*a*) *A stream* :—A stream is a mass of water having a general movement of translation. It is generally bounded laterally by solid substances which form its channel, closed channel like a pipe, or open channel like a canal or river.

The ' axis ' of a stream is a line centrally situated and parallel to the direction of flow.

The stream issuing from an orifice or pipe is called a ' jet ', that falling from a weir, a ' Sheet '. A stream bounded by water is called a current.

(*b*) *Eddy* :—An eddy is the term applied to a mass of water, whose particles have movements irregular and rotatory. It may be of large size and slow motion, or of smaller size but swifter motion. The capacity of eddies for damage depends on their size and intensity and on their position in relation to works. It will be sufficient to consider only large eddies of great intensity and to recognise two classes. (*i*) confined and (*ii*) free. A confined eddy is one confined in contact with a work such as a guide bank, by outside forces, and such eddies are very destructive. A free eddy is one, which will be free to move away from immediate contact with a work such as a guide bank which will otherwise hinder its development. The reader will do well if he watches the motion of water in rivers near protective bunds, spurs, piers, abutments and other works protruding in the river stream, both in clear weather and floods.

(*c*) *Whirlpool* :—This is an eddy of large diameter and high velocity such as caused by the main stream of a large river, running approximately at right angles across the end of a spur, which may be above water, submerged or partly submerged. These whirlpools have been noticed at Sara on Ganges river as 800 feet in diameter and 170 feet in depth below low water-level. There is no reason why they should not be larger and deeper.

On the up stream side of the famous Railway Suspension bridge at Sukkur in Sindh on tho river Indus, there are dangerous whirlpools during floods and the wise ancients put up some sort of a lighthouse on a projecting rocky natural spur in the river to give warning to the boats carrying merchandise and passengers, to be away from the whirlpool. This light is lit up day and night even this day and the rocky spur with its buildings is regarded as a place of pilgrimage by the local inhabitants and is called " Jindo Pir " or an ever alivo saint.

(*d*) *Swirl* :—This is a term applied to moving eddies, the digging effect of which is naturally limited.

(*e*) *Smooth flow* :—Smooth flow is water in motion without whirlpools, eddies, or even swirls and it is only to be found alongside a stone faced guide bank of which the apron is evenly subsided or in the open river.

(*f*) *Turbulent flow* :—Turbulent flow may be described as an on-flowing current, with whirlpools and eddies, caused by the sudden deflection of the stream from its natural course, due to obstruction of a fixed structure such as a protection bank; very deep scour may be expected alongside such a bank specially when the stream of greater curvature is compelled to take the curvature of the protection bank The extent of scour in such a case appears to depend on the direction of the stream impinging on the bank with reference to the alignment of the protection bank and also on the discharge of the river.

(*g*) *Scour* :—Scour is of three kinds. Normal caused by straight and unobstructed currents; abnormal caused by deflected but nevertheless on-flowing currents and by the river running tangentially to a curve and forming an eddy where river leaves the curve of the protection bund; extraordinary, caused by whirlpools, set up in non-forward moving water.

See page 50 of the Journal of the Institution of Engineers (India) Vol. 26, No. 4, June 1946.

8. Pressure Head of a Liquid

In all questions of Hydraulics, it is convenient to express the intensity of the pressure of water in feet of water; that is in terms of the intensity of pressure of the column of water one foot high upon its base, as an unit. A pressure so expressed is called a head of pressure.

One foot of column of water upon a base of one sq foot = one cubic foot = 62·4 lbs. = 62·4 lbs. on one sq. foot, or $\frac{62 \cdot 4}{144} = 0 \cdot 433$ lbs. per sq. inch.

In stating the pressure or head of a particle of water, it is usual not to include the atmospheric pressure which acts equally in all directions on the particle and neutralises itself.

The pressure of the atmosphere at the earth's surface is due to the weight of the column of air above. The pressure of the atmosphere is measured by the height of the column of liquid it will support. This will vary slightly according to the amount of moisture in the atmosphere, the average value may be taken as 14·7 lbs. per square inch, which is equivalent to a static head of 34 ft. of water.

The pressure of water is measured by some type of gauge. A gauge registers the pressure above atmosphere, and the pressure thus measured is termed gauge pressure. To convert gauge pressure to absolute pressure, the reading of the barometer must be added.

If the pressure of the water is below atmospheric pressure, it is measured by means of a vacuum gauge, which gives the amount the pressure is below atmosphere; this must be subtracted from the atmospheric pressure in order to obtain absolute pressure. Thus if the reading of the vacuum gauge is 24 feet of water, the absolute pressure will be $34 - 24 = 10$ feet.

The *atmospheric pressure*, at the level of the sea varies from 32 feet to 35 feet of water, and diminishes at the rate nearly of $\frac{1}{100}$th part of itself for each 262 feet of elevation above that level.

9. Volume and Velocity of Flow.

A stream of water flowing in a channel may be conceived to be made up of a great number of fluid threads, flowing more or less parallel to one another. These threads do not all flow with equal velocities, partly owing to the direct frictional resistance of the boundaries, but mainly because the roughness of the boundaries sets up eddies whereby the fluid filaments cross one another and thus have their velocities modified. The actual motion of a stream is very complex; and there is no theory that takes account of the actual motion of every thread.

Velocity, or the rate at which a particle of water moves along in a stream, usually measured in feet per second, at a given point varies from moment to moment in magnitude and direction. It has been observed that average velocity for a period of, say, a few minutes, is constant.

Suppose the average velocity of each thread in the cross section is determined and that V feet per second is the mean velocity of all such velocities, the discharge Q in cubic feet per second, flowing through the area A sq. feet of the cross section of the stream is

$$Q = A.\ V \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.$$

The motion conceived above, in which the cross section of the stream is divided into very small areas, each of which is the section of a fluid thread, is called *Stream Line Motion*. If each fluid thread or stream line be supposed to have an unchanging velocity, it occupies a fixed position in space and the motion of the stream is termed " a steady motion ".

In any length of stream, in which the flow is steady, and in which no water is lost or gained, the discharges at all cross sections are equal ; in other words, the mean velocity at any cross section varies inversely as the area of the section.

When the shape of the channel in which the liquid moves is nearly regular, the section of the moving liquid also apparently appears as constant, although great changes in the internal arrangement of the particles have taken place.

The greatest velocity in a flowing stream is not at the surface but at some distance below it. The velocity at bottom and sides in contact with the wetted perimeter, is the smallest. Thus the planes of water slide against each other and when a particle slides past another, vacuum is formed and to fill up which, particles from upper layer rush in and collision bombardment takes place, resulting in loss of energy.

The relative velocity of particles, in a vertical plane, in a moving stream of liquid approaches the form of a parabola, the vertex being at a depth below the surface.

10. Principle of Continuity

If any length of a stream be taken with fixed boundaries, this length will generally remain constantly filled and the inflow will be equal to the outflow. If A and A^1, are the areas of two cross sections of the stream, V and V^1, are the mean velocities at those sections, the inflow to the water space between A and A_1 is A V cubic feet per second and outflow is $A^1\ V^1$ cubic feet per second. By the principle of continuity these are equal,

$$A\ V = A^1\ V^1$$

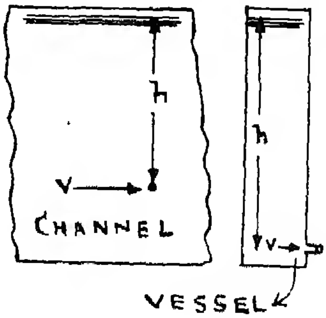
$$\therefore \frac{V}{V^1} = \frac{A^1}{A} \ldots\ldots\ldots\ldots\ldots 4.$$

or the velocities are inversely as the areas. If the bed of the stream has a varying slope, the velocity will be greatest where the slope is steepest, and therefore the cross section the least.

By Dynamics, V, the velocity of moving water is $\sqrt{2\,gh}$:

$\therefore$ h, called velocity head, is $\frac{v^2}{2g}$

Fig. 13B



11. Irregular character of motion

Sit by the side of a river in flood or a big canal, flowing full and you will observe the following phenomena.

The free surface oscillates, more at the sides than at the centre. The motion of water is irregular, the fluid particles do not move in the stream lines, but cross each other. The stream is a mass of small eddies. The irregularities increase with the roughness of the channel and with the velocity of the stream, specially in open channels. Notwithstanding the above facts, the average values of water-level and velocities obtained in successive periods of time of longer duration are more or less constant.

The irregularity of the motion of water renders the theoretical investigation of flow extremely difficult.

12. Contraction and Expansion

A body can not, without coming to rest or describing a curve, change its direction of motion.

Fig. 14 shows when water enters an aperture from a large vessel or a tank.

E. F. is the contracted stream, eddies at sides. Dead pockets or still water at the corners.

Fig. 14

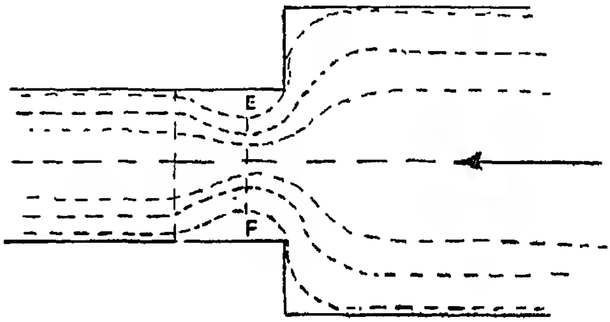
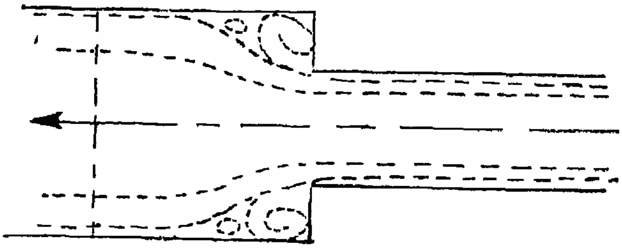


Fig. 14A is an abrupt enlargement. The stream expands generally, with eddies in the corners.

Similar phenomena occur at abrupt bends, bifurcations and junctions.

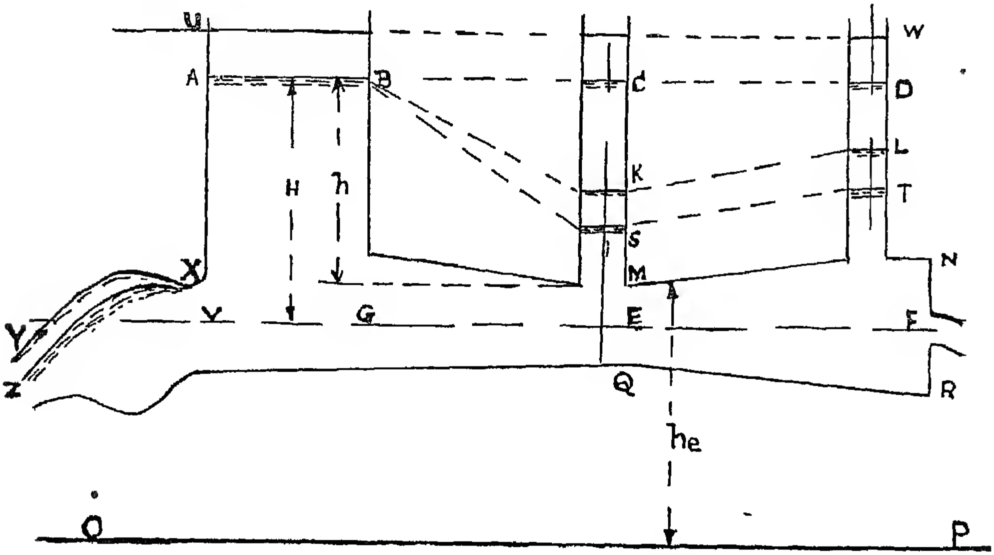
Fig. 14A



13. Bernouilli's Theorem

The figure 15 shows a reservoir of still water, and a pipe with two pressuro tubes and an outlet F at the end of the pipe and an outlet V at the side of the reservoir. Suppose the above two outlets are closed. The water in the reservoir and the two tubes is at the same level, Line A B C D. The Head over any point is its depth below the plane A B, sometimes called the plane of chargo. The pressure at this point is as the head. $P = W H$. or $H = \frac{P}{W}$, H being called the Pressure Hoad.

Fig. 15



Every particle of water in the reservoir possesses the same degree of potential energy. The particle at depth H possesses energy in virtue of its pressure, the particle at surface level possesses energy in virtue of its elevation.

Let an orifice be opened at F, so that water flows along the pipe GEF, and let the reservoir be large, so that the water in it has no velocity and the surface AB is unaltered. The pressure in water flowing in the pipe GEF is reduced and the water-levels in the tube fall to points K & L. The heights KM & LN are the pressure heads at M and N.

The tubes are called pressure-columns and line BKL is Hydraulic gradient line. Let P be the pressure at M and h_p the pressure head. Then $h_p = \frac{P}{W}$. Let V be the velocity at the pipe at point M and let $h_v = \frac{V^2}{2g}$. Thus h_v is the 'velocity head'. It is the height through which a body falls under the influence of gravity in an unresisting medium in acquiring the velocity V, or the height to which it could be made to rise by parting with its velocity. Suppose there is no resistance to the motion of water so that no energy is consumed in overcoming them.

Thus by the law of conservation of energy, the total energy of any moving particle of water remains as before.

Whatever is lost as pressure, is gained as velocity. The head CK lost in pressure is the velocity head h_v. Thus $h = h_p + h_v$ ....4A or pressure head added to the velocity head is the hydrostatic head. This equation due to Bernouilli is the basis of all theoretical formulae. It applies to any point in the pipe.

It has been seen that the pressure at M is as the height KM. Assume that velocities at all points in the cross section MQ are equal. Let H_p and H_v be the pressure head and velocity head at E, then $H = H_p + H_v$; $h = h_p + h_v$.

Since the velocities are equal, $H_v = h_v$; therefore $H_p - h_p = H - h$ or the change in pressure in passing from M to E is the same as it was when there was no flow.

The pressure head at E is KE, and the pressure at any point in the cross section is as its depth below K.

Let OP be a datum line and let h_e be the elevation of any point M above OP. Then $h + h_e$ is constant for all points in the system. Therefore

$$h_p + h_v + h_e = K \ldots\ldots 5.$$

Where K is a constant.

This is Bernoulli's theorem more fully stated. The total energy possessed by a particle of water is the sum of energies due to its pressure, velocity and elevation.

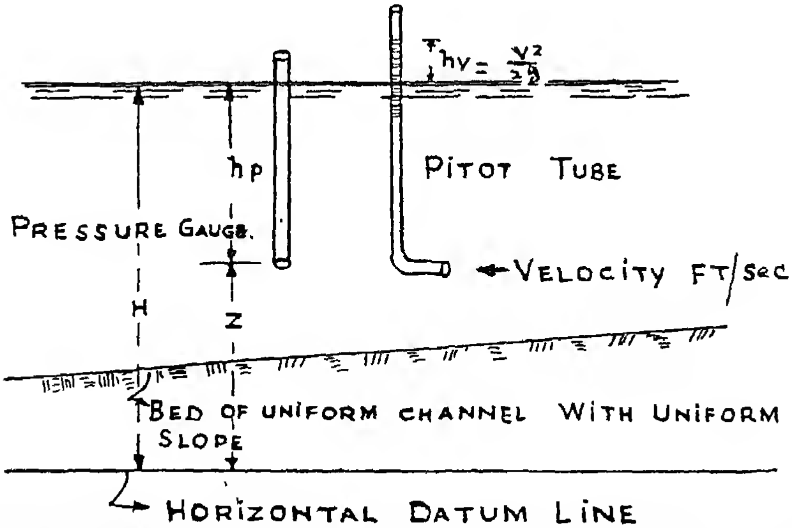
If instead of a pipe we consider an open channel XY, the results obtained will be the same as before. If pressure columns were used, the water in them would not rise above surface XY. At each point in the surface, the pressure head is equal to the hydrostatic head. If the velocities at all points in a cross section are assumed to be equal, the law of change of pressure with the depth is the same as before.

Since the area NR is greater than MQ, the velocity is less and pressure greater. Thus from K to L there is a rise in the Hydraulic gradient. Similarly in the open stream, there is a rise when the sectional area is increasing.

The pressure in a body of flowing water can never be negative, as the continuity of the liquid would be broken. (Page 10 of Hydraulics by Bellasis, Rivingtons, London 1903).

BERNOUILLI'S EQUATION SIMPLIFIED

Fig. 16



The above sketch shows a uniform channel with a uniform slope and uniform discharge. There is a Pitot's tube with an impact orifice at lower end. The water rises in this tube above the free surface of the water in channel by height h_v, which is $= \frac{V^2}{2g}$: h_v can be read on the tube or can be calculated from V, which can be ascertained by a current meter. h_p is pressure gauge, its lower end can be ascertained by a level.

The whole energy of a unit mass of water at a point z above the Datum plane is constant.

$h_p + Z + h_v =$ constant : This is Bernouilli's equation .... 6.
Loss of head due to friction is for the present neglected.

In the above channel, water flows under the influence of gravity. There is a fall in the bed. This fall causes the flow of water, the velocity of flow and overcomes the resistance on account of contact of water with bed and sides of channel.

The free surface of water is parallel to the bed and is the Hydraulic gradient line.

The two tubes shown above are so near each other that the water surfaces in the same may be assumed to be at the same level.

14. Miscellaneous

(*a*) *Loss of head from Resistances.*

Practically a certain amount of head h' is expended in overcoming resistances, due to the friction of water on its channel and to the internal movements of water, so that the total head diminishes in going along the stream in the direction of flow. In other words, the pressure head and velocity head do not together equal the hydrostatic head. The difference is the 'head lost'. The actual water levels in practice would be S, T and CS, DT would be the total losses of pressure head upto the points M and N (See Fig. 15). As the head is lost, the work which water is capable of doing in virtue of its elevation, pressure and velocity is diminished. If h' is the head lost by resistance between two cross sections, then

$$h' = h - \left(\frac{V_2^2 - V_1^2}{2\,g}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ 7.$$

or the head lost is equal to the fall in surface or line of gradient less the increase in velocity head. The same is true of open channel. The surface would be XZ instead of XY (Hydraulics by Bellasis, page 11).

If the motion of water is impeded by friction, there is an additional loss of head, bearing to the velocity of flow a certain proportion : water can flow only from a higher point to a lower point in a channel. Let this difference of level be h. The mean velocity of flow is $= \sqrt{2gh}$: Loss of head due to friction, depending on the figure and dimensions of the channel and openings traversed by the stream and other circumstances, bears a certain proportion to the height (h_v), due to the velocity of flow.

Let the loss of head due to friction be called h_c. $h_c = F\,h_v = F\dfrac{V^2}{2g}$, F being a factor determined by experiment, expressing proportion which the loss of head by friction bears to the height due to velocity.

The total fall in the bed of the channel, $h = h_v + h_c = \dfrac{V^2}{2g} + F\,\dfrac{V^2}{2g} = (1 + F)\,\dfrac{V^2}{2g}$ (Rankins' Civil Enignecring, page 675, Edition 1877).

Quantity of flowing water for a certain thickness along the perimeter, (depending upon the physical build of the surface), constantly loses its kinetic energy causing turbulence in the mass of water. Greater the remaining mass of water, less is the total effect of turbulence. That is why the term $\dfrac{\text{area}}{\text{wetted perimeter}}$ = Hydraulic mean radius, automatically comes in the equation of velocity.

It is known that viscous resistance in the mass of liquid, surface tension and such other factors play a great part in determining the velocity of flow.

The theory of modern fluid mechanics teaches that a very thin layer of fluid in laminar motion exists near the solid boundary of the channel. This laminar flow exists even when the flow is turbulent, *i.e.*, ultra critical.

While swimming in the canals, one notices how the momentum or shock of flowing water acts on the body. This fact also enters the velocity theory.

(b) *Radiating and circular currents.*—(Bellasis, page 294).

Suppose water to be supplied by the pipe AB (Fig. 16A), and then to flow out radially between two parallel horizontal surfaces CD and EF, whose distance apart is d. Of radii R_1, R_2, let R_2 be the greater, and let the velocities be V_1, V_2 and pressures P_1, P_2. Since the discharges past all vertical cylindrical sections are equal, therefore $\frac{R_1}{R_2} = \frac{V_2}{V_1}$.

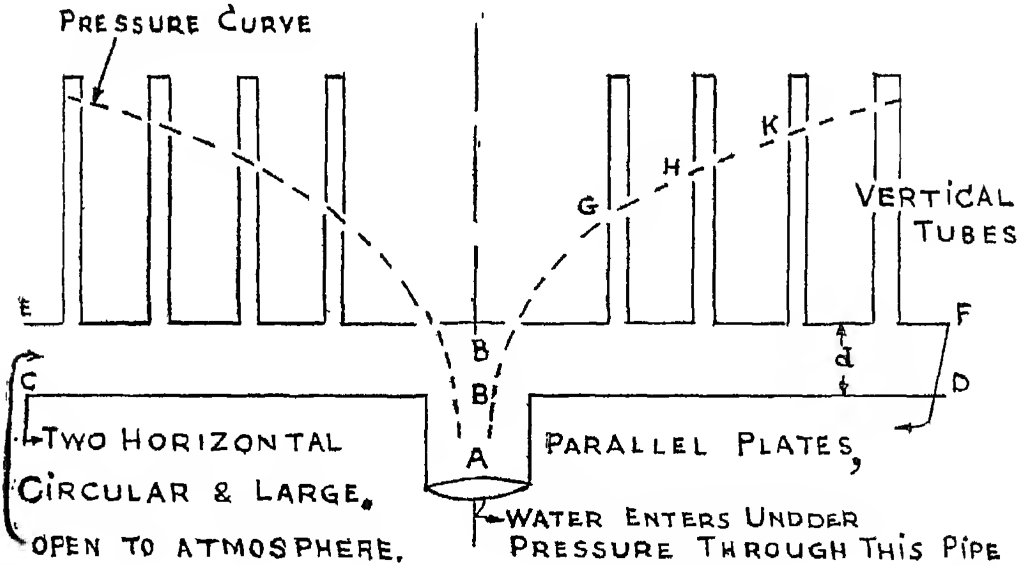
Also since by Bernouilli's theorem the hydrostatic head is $H = \frac{P_1}{W} + \frac{V_1^2}{2g} = \frac{P_2}{W} + \frac{V_2^2}{2g} = \frac{P_2}{W} + \left(\frac{R_1^2}{R_2^2} \times \frac{V_1^2}{2g}\right)$

Therefore $\frac{P_1}{W} = H - \frac{V_1^2}{2g}$

And $\frac{P_2}{W} = H - \frac{V_1^2}{2g}\frac{R_1^2}{R_2^2}$

or the heights in pressure columns increase from the centre outwards and tend to reach, though never reaching, the value H.

Fig. 16A



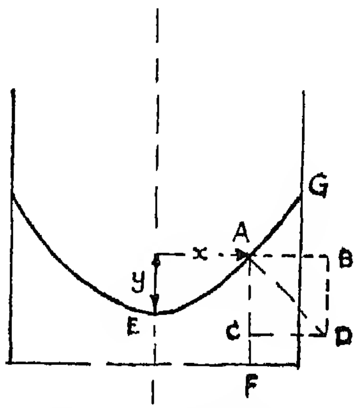
If the water flows inwards and passes away by the pipe the law is the same. A curve through the points G, H, K, etc., is known as Barlow's curve.

In a vessel which, with its contents, is revolving about a vertical axis with angular velocity α, the forces, acting on a particle A whose velocity is V, are its weight W or AC, acting vertically, and a horizontal centrifugal force $W\frac{V^2}{gx}$ or $W\frac{\alpha^2}{g}x$ or AB.

The water surface takes a form normal to the resultant AD of the above, that is, the angle DAC is

$\tan^{-1}\frac{\alpha^2}{g}x$ Hence $\frac{dy}{dx} = \frac{\alpha^2}{g}x$

Fig. 16 B



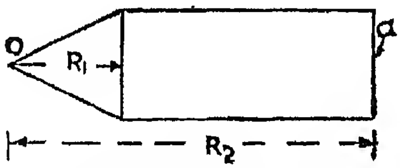
Integrating, $Y = \frac{\alpha^2}{2g}x^2$, or the curve EA is α parabola with apex at E, since $V = \alpha x$, therefore $Y = \frac{V^2}{2g}$ or the elevation of any point above E is the head due to its velocity of revolution. The theoretical velocity of efflux from an orifice at F or B is that due to a head AF or GB.

A similar condition occurs in a mass of water driven round by radiating paddles. In either case the condition is termed a "forced vortex". Questions connected with the pressure in a radiating current or in a forced vortex enter, though not to a very important degree, into the theories of certain hydraulic machines.

In a centrifugal pump, the pressures in the pump wheel follow the law of radiating currents, while those in the whirling chamber outside the wheel, depend on the law of the forced vortex.

(c) *Centrifugal Head impressed on revolving liquid.*—(Lewitt).

Fig. 16 C



A rotating fluid is called a vortex. If the fluid is rotating freely without any external forces being impressed upon it, it is called a free vortex. An example of a free Vortex is the whirlpool formed in the emptying of a wash basin having a central outlet.

If the liquid is rotated by an external force, the vortex is termed a forced vortex. A forced vortex will have a centrifugal head impressed on the liquid, caused by its rotation.

Referring Fig. 16c, imagine an arm containing water to be rotating in a horizontal plane about the centre O and with an angular velocity of ω. Let the arm be full of water between a radius of R_1 and R_2 and let the cross sectional area of the arm be a.

Total centrifugal force impressed on whole of rotating water

$$= \frac{w\alpha\ \omega^2}{2\ g}\left(R_2^2 - R_1^2\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots(7d)$$

$$= \frac{w\alpha}{2g}\left(v_2^2 - v_1^2\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots(7e)$$

If v_1 = tangential velocity at radius of $R_1 = \omega R_1$

v_2 = do. do. do. $R_2 = \omega R_2$

Intensity of pressure at end of arm due to centrifugal force

$$= \frac{w}{2g}\left(v_2^2 - v_1^2\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots(7f)$$

Centrifugal head impressed $= \frac{P}{W} = \frac{V_2^2}{2g} - \frac{V_1^2}{2g}$ ....(7g).

Thus the centrifugal head impressed on a revolving fluid is the difference between the tangential velocity heads.

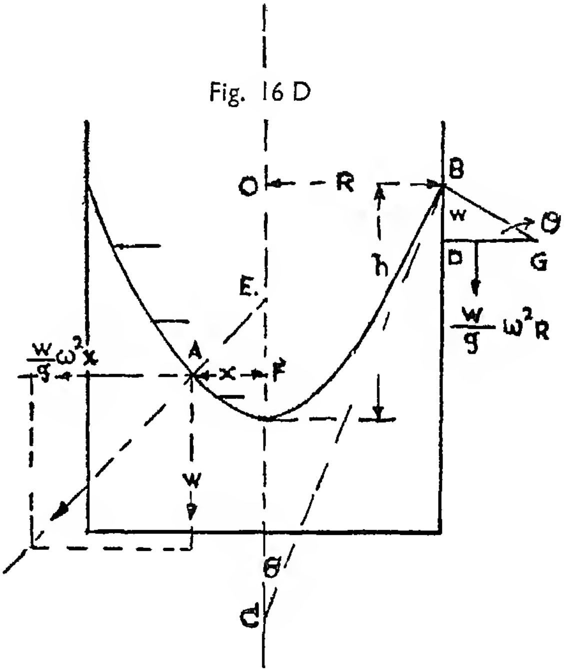
This is the principle of the centrifugal pump, which obtains its lifting power from this head.

(d) Revolving cylinder of liquid. (Lewett).

Consider a cylinder containing a liquid to be revolved about a vertical axis OC (see fig. 16D). The surface of the liquid will take the shape of a paraboloid as shown. This is another example of a forced vortex.

Consider a small particle of the liquid at the point A on the surface. Let W be the weight of the particle. It will be in equilibrium under the action of three forces . the weight, the centrifugal force, and the pressure.

Fig. 16 D



Let ω = angular velocity of cylinder.
x = radius of particle.

Centrifugal force on particle = $\frac{W}{g}\omega^2 x$ .... (7h).

The centrifugal force will act horizontally outwards, and the weight vertically downwards. The resultant of these two will be opposed by the pressure of the fluid. As the latter must act normal to the surface, it follows that a tangent to the surface at A will be at right angles to the resultant of the centrifugal force and the weight. It follows from fig. 16D that

$$\frac{EF}{x} = \frac{W}{\frac{W\omega^2 x}{g}}$$ (similar triangles). Therefore, EF $= \frac{g}{\omega^2}$ and is therefore a constant.

As EF is the subnormal of the liquid surface, the shape of the surface is a paraboloid.

Consider the liquid at B. Let R = radius at B. θ = angle of inclination of surface at B to vertical. h = height of paraboloid. Consider the similar triangles BDG and BOC : $\frac{BD}{DG} = \frac{BO}{OC}$

or $\dfrac{W}{\dfrac{W}{g}\omega^2R} = \dfrac{R}{2h}$, as OC will be twice the height of paraboloid. Therefore $h = \dfrac{\omega^2R^2}{2g} = \dfrac{V^2}{2g}$,

(7i) where V is the tangential velocity at the point B.

It will be noticed that h is a direct function of the square of the speed of rotation. This principle is made use of in a type of speedometer used in Engine Testing. A cylindrical glass vessel containing a liquid is rotated by the engine, the speed of which can be estimated by the height 'h' of the paraboloid formed in the vessel.

EXAMPLE :—

Speed of a steam engine is required to be measured. A glass cylinder containing oil is rotated on a vertical axis by the engine and is geared at thrice the speed. If the paraboloid formed by the rotating liquid is 4 inches high with a maximum radius of 2 inches, find the number of revolutions per minute made by the engine. (Lowett).

See equation No. 7i.

Here $h = 4$ inches $= 1/3$ foot.

$R =$ Radius 2 inches $= 1/6$ foot.

$\omega =$ angular velocity of the cylinder in radians per second.

$h = \dfrac{\omega^2R^2}{2g}$ Then $\frac{1}{3} = \dfrac{\omega^2}{64\cdot4}\left(\frac{1}{6}\right)^2$

$\omega = 27\cdot84$ radians per second.

Speed of cylinder $= \dfrac{\omega}{2\pi} = \dfrac{27\cdot84}{2\pi} = 4\cdot40$ revolutions per second.

Speed of engine $= \dfrac{4\cdot40 \times 60}{3} = 88$ revolutions per minute.

CHAPTER V.

HYDRO-DYNAMICS

15. LAWS :—This is a branch of science which deals with laws regulating the flow of water in pipes, channels and rivers ; also in water-pumps, turbines and other hydraulic machines. The laws are :—

(*i*) If the motion is rectilinear and uniform and if the effects of eddies produced by the roughness of the boundaries of streams be neglected, the pressure at any point is the same as if the fluid were at rest.

(*ii*) If the fluid particles take the same acceleration, which they would have if independent, the pressure is uniform. Thus in a jet falling freely in the air, the pressure throughout any cross section is uniform and equal to the atmospheric pressure.

(*iii*) Pressure, produced by a stream of water when its velocity or direction of motion is altered, is called dynamic pressure. This is, of course, entirely different from static pressure. Let V, A, and Q be the velocity, sectional area, and discharge (cusecs) of a stream and w be the weight of one cubic foot of water.

The volume discharged Q in cusecs $= A \times V$

Momentum of Q (Cusecs) $= Q \dfrac{WV}{g} = \dfrac{AV^2 W}{g}$ foot-lbs. Thus the force F, which acting for one second, will produce or destroy this momentum is

$$F = \frac{wAV^2}{g} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 7\ a.$$

On this principle, the pressures developed in various practical cases can be ascertained. This subject is also partly treated in chapter iv.

The kinetic energy = Mass $\times$ V^2.

Force = Mass $\times$ acceleration.

The height through which a body falls vertically in t seconds $= \frac{1}{2} gt^2$.

CHAPTER VI.

DISCHARGES THROUGH SMALL ORIFICES

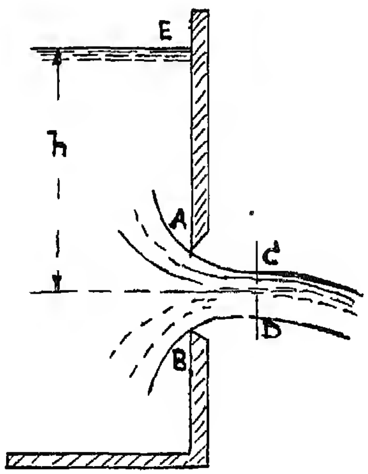
16. Preliminary

An apperture means an opening. If it is in the side of a vessel, full of water or in a thin wall, so that it is bounded on all sides by something solid, it is called an orifice.

When an orifice has its sides produced to the free surface of water, so that its top edge lies in the said free surface, it is called a notch. In this case the difference between the heads at upper and lower edges is most pronounced.

17. Coefficient of Contraction

Fig. 17



If the orifice be in a thin plate, or if the edges of orifice be bevelled off sharp, the cross section of the jet attains a minimum value on line C D, at a distance from the plate, equal to half the diameter of the orifice, or half of the least diameter if the orifice is of elongated form.

The contraction takes place because the particles of water while moving towards the orifice, can not, due to the principle of Inertia, change their course suddenly. Hence the paths of the particles are curved as shown in figure 17. There should be a clear margin round the orifice to ensure contraction.

The minimum section of the jet on line C D is called "Vena Contracta". The ratio of the sectional area of "Vena Contracta" to the area of the orifice is called coefficient of contraction, denoted by C_c, usually about 0·63, as determined.

The contraction depends upon the shape and size of the orifice and on 'h'.

'h' is the head or distance from the centre of the orifice to the free surface of water in the vessel or tank, which is big enough to maintain the level of the free surface constant, when the discharge from the orifice takes place. Part of 'h' is consumed in overcoming friction between water and sides of the orifice.

18. Velocity of Discharge

Fig. 18

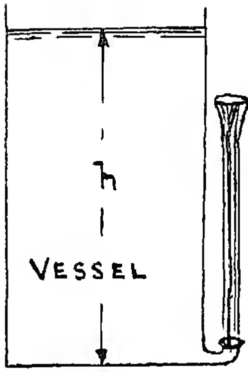


Fig. 18 A

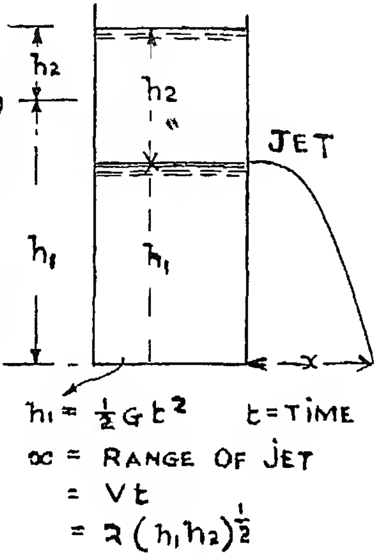


Fig. 19

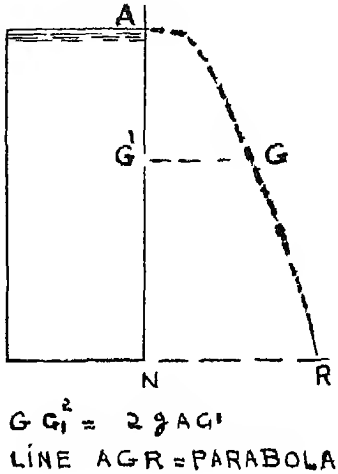


Figure 18 shows an orifice fitted with a vertical jet. Pressure at bottom of vessel, $P = wh$. Velocity of particle of water while coming out of the orifice is as if it fell through a height h. This is called theoretical velocity and according to Dynamics

$V = \sqrt{2gh}$ ....8.

When the water shoots up vertically from the jet, it goes to the height of h^1 from the bottom of the vessel, the height h^2 ($= h - h^1$) being lost due to friction and resistance*.

Since $V = \sqrt{2gh}$, $h = \frac{V^2}{2g}$, the velocity head so called.

When the particle of water comes out horizontally its path while falling to the ground is a parabola as shown in fig. 18A.

In figure 19, if N R be the theoretic velocity at point N for a particle of water through a horizontal orifice and if AGR is a parabola with axis A N and parameter equal to 2g, then the ordinate $G G^1$ from any point G^1 represents the velocity of water through the orifice at G^1.

$$\therefore \frac{(N R)^2}{A N} = \frac{(G G^1)^2}{A G^1} = 2g = \text{constant.}$$

19. Coefficient of Velocity

Actual velocity 'Va' differs from the theoretical velocity 'Vo' by a small amount. The ratio of $\frac{Va}{vo}$ is called coefficient of velocity, which is determined by experiment and is nearly constant for different heads. Its mean value is 0·97.

* Credit for this explanation goes to Toricelli, a Philosopher and Inventor of Barometer in 17th Century. He was pupil of Gallilo, an Italian Astronomer.

20. Coefficient of Discharge

Theoretical discharge through an orifice is equal to (the area A of the orifice) $\times$ (the theoretical velocity due to the head h). But in practice owing to reduction in velocity and to the contraction of the jet, the actual discharge is less than the theoretical discharge. The actual discharge can be ascertained by measuring the quantity of water discharged through the orifice in a gauge basin in a given time, care being taken to keep the head h of water in the vessel constant by adding more water from time to time. The ratio of actual discharge to the theoretical discharge is called coefficient of discharge. It is equal to coefficient of contraction multiplied by coefficient of velocity, the product being termed C. This is generally $0 \cdot 64$ for an orifice in a thin plate. $Q = C\,A\,\sqrt{2gh}$, h being measured from centre of orifice to water surface.

Example:—Find the head of water which will ensure a discharge of 8 C. ft. per second through an orifice of 6 inches square in a thin plate.

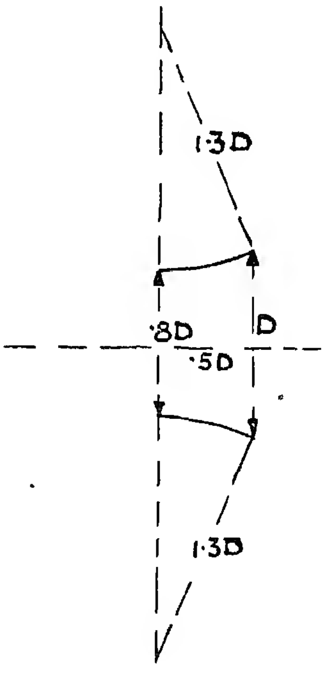
$Q = C\,A\,\sqrt{2gh}$, where $Q = 8$, $A = \frac{1}{4}$, $C = 0 \cdot 62$ ft.

$\therefore\;\; 8 = 0 \cdot 62 \times \frac{1}{4} \times 8\,\sqrt{h} \;\; \therefore \sqrt{h} = 6 \cdot 45 : h = 41 \cdot 6.$ ft.

Suppressed contraction:—Since contraction is caused by the convergence of fluid threads, any circumstance which tends to diminish that convergence, such as the application of an internal rim to border of the orifice, or near approach of the orifice to the bottom or sides of the vessel, will tend to increase the coefficient of discharge. The expression $C = 0 \cdot 62 \left(1 + 0 \cdot 14 \frac{n}{m} \right)$ 9A

will meet the case, n being the length of the perimeter of orifice where contraction is suppressed, and m being the perimeter of the orifice. The coefficient is also modified by the application to the orifice of shoots and mouth pieces. In a bell-mouth the contraction must be complete whatever the clear margin (clear space round the orifice) may be.

Fig. 20



21. Bell-mouths

If the orifice be of the form of the Contracted vein, or in other words a simple bell-mouthed tube is made of the shape of the jet, issuing from an orifice in a thin wall, the whole of the contraction occurs within the orifice, and if the area of the orifice be measured at its smaller end, the coefficient of contraction is unity. Hence coefficient of discharge $C = 1 \times$ coefficient of velocity $= 0 \cdot 97$. The entrances to pipes from reservoirs are often bell-mouthed to avoid the contraction, and consequent loss of head which would otherwise occur: for *Bell-mouths*, discharge $Q = 0 \cdot 97\,A\,\sqrt{2gh}$.

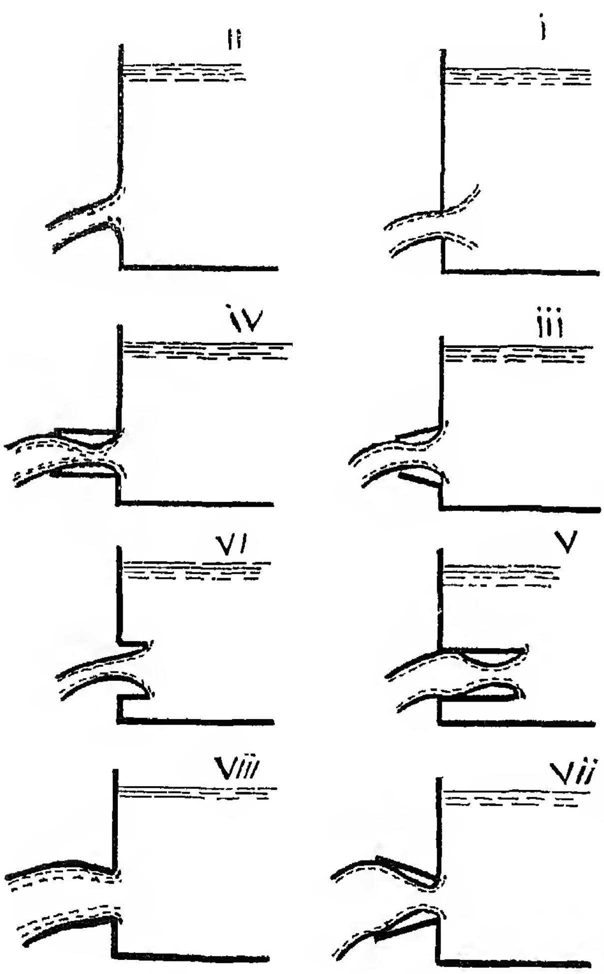
22. Mouth Pieces

If a cylindrical tube of length not less than $1\frac{1}{2}$ times the diameter of the orifice, be applied externally to the orifice, the jet after contraction again fills the tube, and coefficient of discharge attains the value $0 \cdot 82$ and the discharge increases. Pressure at Vena-Contracta decreases and the effective head causing the flow increases.

The principal kinds of orifices or short tubes met with in practice, with their average coefficients are as follows :—

SKETCH.

Fig. 21



Description of Orifice or Tube.	Average Coefficients for complete contraction.		
	Cc	Cv	C
(*i*) Orifice in thin wall ..	·63	·97	·61
(*ii*) Bell-Mouthed Tube ..	1·0	·97	·97
(*iii*) Convergent Conical Tube ..	·98*	·96*	·94*
(*iv*) Cylindrical Tube ..	1·0	·82	·82
(*v*) Inwardly Projecting Cylindrical Tube ..	1·0	·72	·72
(*vi*) Borda's Mouth Piece	·52	·98	·51
(*vii*) Divergent Conical Tube 5°	—	—	1·46†
Divergent Tube with Bell-Mouth ..	1·0	2·0	2·0‡

* For the smaller end of the Tube. Angle of Cone is 13°.
† For the Smaller end when angle is 5° — 6′.
‡ For the smallest Section: (see hydraulics by Bellasis page 43.)

23. Velocity of Approach

In Fig. 17, article 17, it is assumed that the free surface of water is still and that there is no velocity of approach to the Orifice : Suppose there is velocity of approach Va ; then from equation 8 the velocity head $ha = \frac{Va^2}{2g}$: Let 'h' be the head over the centre of orifice, some little distance back from the orifice, where the water is calm and undisturbed. The actual velocity of discharge =

$$C\sqrt{2g\left(1+\frac{ha}{h}\right)} \quad \ldots\ldots\ldots\ldots 10.$$

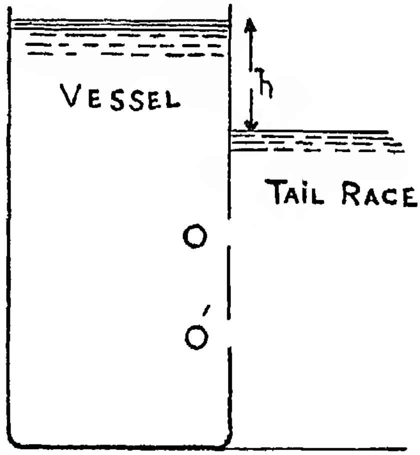
This is approximate.

24. Submerged Orifices

In this case, the surface of tail water is above the top of Orifice (Fig. 22). The above formulæ still hold good but 'h' is the difference in level between the surface of the reservoir and tail water and is the same for orifice O or O^1

$Q = C\,A\,\sqrt{2gh}$

Fig. 22



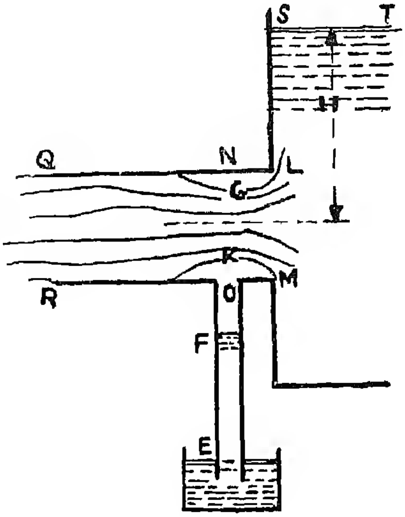
25. Short Pipes

As the cylindrical mouthpiece is gradually increased in length, so as to become a short pipe, the frictional resistance increases and the coefficient diminishes as follows :—

Length in diameters.	2	5	10	25	50	100	150	200	250	300	1000
Coefficient of Discharge. New pipes.	0·82	·79	·77	·71	·64	·55	·49	·44	·41	·38	·216

In a cylindrical tube Fig. 23 the jet contracts, but it expands again, fills the tube and issues "full bore".

Fig. 23



Sectional area GK, as in a simple orifice in a thin wall, is 0·63 times the area LM; but the velocity at GK is greater than $\sqrt{2gh}$ and the discharge through the tube is greater than that of an orifice of the size LM.

When there is flow, partial vacuum is formed at N & O. The pressure at G & K is less than the atmospheric pressure by 0·69wh, provided h is relatively large to the length of the tube: The velocity at GK is also $= 1{\cdot}37\sqrt{2gh}$.

Since there is a partial vacuum at O, water-level would rise in the tube E O, for ordinary heads, to a height $EF = {\cdot}75\ h$ and if the height E O is less than this, water will be drawn up the tube and discharged with the jet. This is the Principle of a jet Pump. The height to which water can be pumped, even if the vacuum is perfect, is limited to 34 feet.

The coefficient of discharge for a cylindrical tube, like that for a simple orifice, increases as the head and diameter decrease.

Coefficient of contraction and velocity and discharge for different orifices are given in article 7. Detailed information is given in books on Hydraulics by Bellasis, Smith Neville, who made a very large number of experiments with orifices, etc.

Example.—Find the discharge per second through a pipe, whose length is 4 feet and diameter 12 inches, the head or depth from the surface of water to centre of pipe being 12 feet.

From the table in article 25, the coefficient may be taken as 0·80.

$$\text{Discharge} = C\sqrt{A\,2gh} = 0{\cdot}80 \times 0{\cdot}785 \times \sqrt{2 \times 32 \times 12}$$
$$= 17{\cdot}4 \text{ C. ft. per second.}$$

N.B.—See Figures 18, 18A, 19: curve described by the jet is a parabola with equation $x^2 = 2gh^1$, 2g being the parameter: $x = vt$, v being actual velocity. $\frac{x^2}{V_2} = t^2 = \frac{2h^1}{g}$ $\therefore V^2 = \frac{x^2g}{h_1}$: Theoretical velocity $= \sqrt{(2gh_2)}$: Coefficient of velocity $Cv. = \frac{\text{actual vel.}}{\text{Theorl. vel.}} = \frac{x}{\sqrt{(4h_1h_2}}$: By actually measuring x, h_1, h_2, many problems can be solved by the help of above equations.

CHAPTER VII.

DISCHARGE FROM LARGE ORIFICES AND NOTCHES

26. Large Orifices

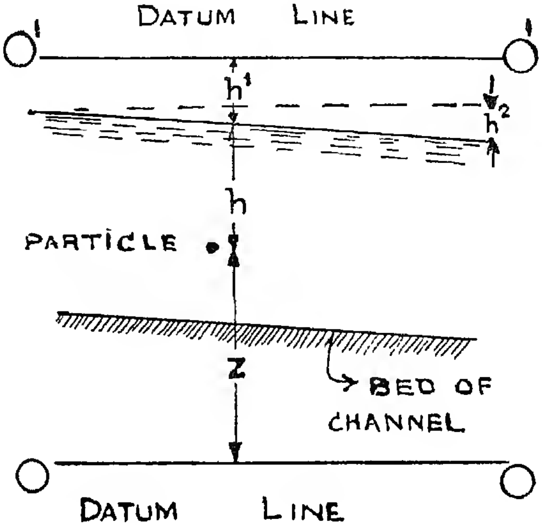
In the last chapter we have dealt with small Orifices, *i.e.*, those in which the head is nearly the same for all the issuing threads. In the case of large orifices, the velocity cannot be taken the same for all the threads of the jet, owing to heads differing widely for the threads at top and bottom of Orifices respectively. However the mean velocity of the threads is nearly the same as the velocity of the thread at centre of the Orifice. Hence the expression for discharge *viz.*

$Q = C\ A\ \sqrt{2gh}$ holds good in this case as well, 'h' being the height from the centre of the Orifice to the water surface.

27. Bernouilli's Theorem

This theorem has been explained in articles 13 and 14. It is repeated here.

Fig. 24



The figure 24 shows a channel with regular bed slope and water flowing down in steady motion. Take any particle with its height above datum o o. Then $\frac{V2}{2g}$, kinetic energy, is the head due to velocity and represents the amount of work to be done by the particle (Say 1 lb. of water) moving with velocity V feet per second. Suppose the particle moves down for one second, pressure energy $\frac{P}{W}$ (Since P = wh) represents the quantity of work done by the particle under pressure P and moving with velocity V.

Z potential energy represents the work to be done by 1 lb. of water falling to the datum line o o. The sum of these energies is therefore the total energy per pound of water estimated with reference to the datum line o o. This is uniform along the stream line.

If the particle moves down to a new position and acquires pressure head = $\frac{P^1}{W}$ and potential head Z^1 and velocity V^1, then $\frac{V_1^2 - V^2}{2g} = \frac{P}{W} - \frac{P^1}{W} + Z - Z^1$ .... 11.

$$\therefore \frac{V^2}{2g} + \frac{P}{W} + Z = \frac{V^{12}}{2g} + \frac{P_1}{W} + Z^1 = \text{constant} \qquad \text{12.}$$

This is Bernouilli's theorem or application of the law of conservation of energy to flowing water.

If the depth of the particle is measured from the datum o^1 o^1, then Z becomes negative and the equation is

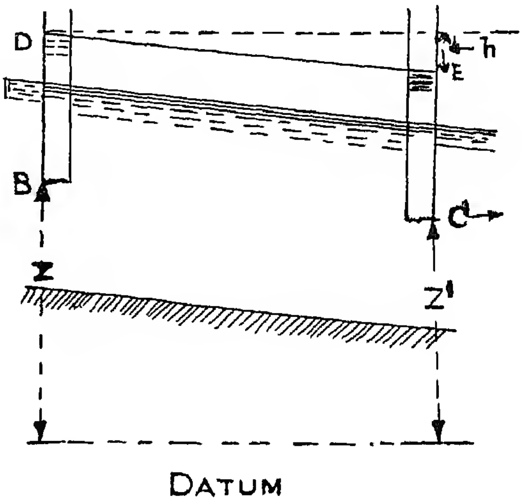
$$\frac{V^2}{2g} + \frac{P}{W} - (h + h^1) = \text{constant} \qquad \text{13.}$$

In a stream line, the fall of surface level between two sections is the difference of heights due to velocities at those sections or $h^2 = \frac{V_1^2}{2g} - \frac{V^2}{2g}$ see Fig. 24.

Venturi meter is designed on the principle of Bernouilli's theorem.

28. Hydraulic Gradient

Fig. 25



Let two vertical tubes be placed so as to meet a stream line at B & C, Fig. 25, the points B and C being at the same depth below the water surface in a channel with regular bed slope and steady flow. The water will rise in the tubes to point D and E, due to pressures at B and C:

B D will be equal to $\frac{P}{W}$ and C E $= \frac{P^1}{W}$,

P and P^1 being pressures at B and C.

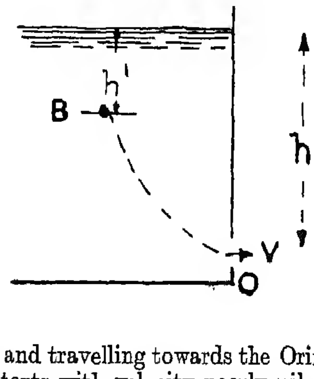
The difference of level "h" between the free surface of water in the tubes is equal to $Z + \frac{P}{W} - \left(Z_1 + \frac{P^1}{W}\right)$.

See equation No. II in article No. 27 above : $h = \frac{V_1^2 - V^2}{2g}$, V_1 and V are velocities of flow at C and B respectively.

This means that fall of surface levels between two sections at B and C is the difference of heights due to the velocities at those sections. The line D E is termed the *Hydraulic Gradient*, but this expression is also used in cases where friction is taken in account.

29. Velocities of Threads Issuing in a Jet.

Fig. 26



The Fig. 26 shows a vessel with water surface constant, water jet coming out of an Orifice near the bottom with velocity V feet per second.

Take a particle of water at point B, anywhere in the vessel, and travelling towards the Orifice. Other particles follow it and form a thread. The particle at B starts with velocity nearly nil. It acquires velocity V at the Orifice O.

At B, the head is "h^1", the pressure is π (atmospheric pressure) + w h^1 and the velocity Zero.

At O, the head is " h ", the pressure is π and Vel. V.
By Bernouilli's theorem.

$$O + \frac{\pi + wh^1}{w} - h^1 = \text{constant} = \frac{V^2}{2g} + \frac{\pi}{w} - h$$

Free water surface being the datum line.

$$\therefore h = \frac{v^2}{2g} \text{ or } V = \sqrt{2gh}$$

Velocity of discharge is explained also in article No. 9.

30. Rectangular Notch

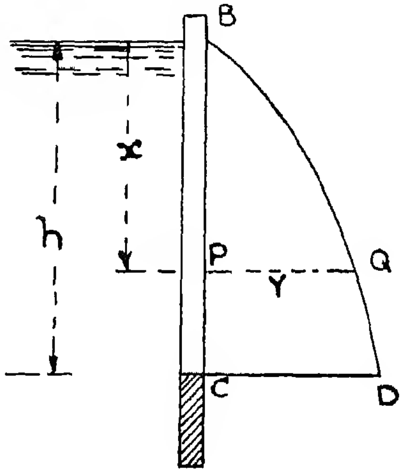
Consider a rectangular notch of length 'L' and of depth B C in the vertical side of a vessel filled with water, the water level being kept constant by the addition of water in the vessel. See Fig. 27.

A fluid thread at a point, χ feet below the water surface, has a theoretic velocity equal to $\sqrt{2g\chi}$.

Lay off P Q horizontally equal to $\sqrt{2g\chi}$, to represent the velocity at P. The outer extremities of all such lines as P Q lie on the parabola B Q D, C D of course being equal to $\sqrt{2gh}$. Since P Q $= \sqrt{2g\chi}$ = Ordinate Y of the Parabola. Then $Y^2 = 2g\chi$, which is equation to a parabola whose axis is B C and vertex at B, the parameter being 2g.

The figure B D C is graphic representation of velocities of all the threads issuing from vertical line B C.

Fig. 27



The mean velocity of all the threads =

$$\Sigma \frac{(P\,Q)}{B\,C} = \frac{\text{area } B\,C\,D}{h} = \frac{\frac{2}{3} h \times \sqrt{2gh}}{h} = \frac{2}{3}\sqrt{2gh}$$

i.e. the mean velocity is $\frac{2}{3}$ of bottom velocity.

The actual discharge is

$$Q = \tfrac{2}{3}\, C\, L\, h \sqrt{2gh} = \tfrac{2}{3}\, C\, L\, h^{\frac{3}{2}} \sqrt{2g} \quad \ldots\ldots 14.$$

C is coefficient of discharge, 0·62 for a thin plate, for complete contraction. This equation is not used for large weirs.

Mr. Francis* takes the value of 'L' in the above equation as L—0. 2 h in case 'L' is more than 3 times 'h', when there are two end full contractions in a notch in a thin plate. Practical examples are gauge boards, tank surplus weirs and barrages across rivers.

*Francis, a Hydraulician of 19th century.

31. Large Rectangular Orifice.

Let 'L' be the length of the orifice, h_1 & h_2 the heads to the bottom and top respectively, Fig 28. The velocity at C $= \sqrt{2gh^1}$ represented by C D.

The velocity at E $= \sqrt{2gh_2}$, represented by E F. Hence the mean theoretic velocity is

$$\text{area } \frac{E\,F\,D\,C}{E\,C} = \frac{\frac{2}{3} h_1 \sqrt{2gh_1} - \frac{2}{3} h_2 \sqrt{2gh_2}}{h_1 - h_2}.$$

The theoretic discharge $= A\,v$

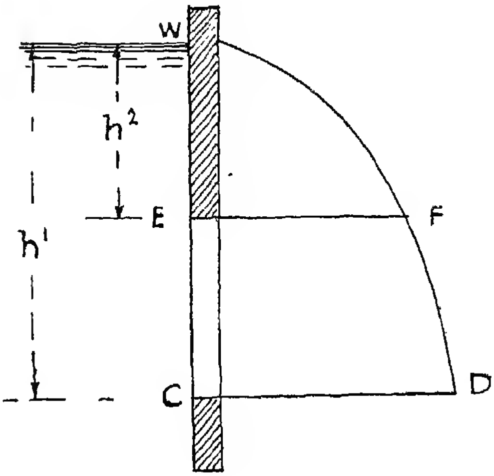
$$= L\,(h_1 - h_2) \frac{\frac{2}{3}\sqrt{2g}\,(h_1^{\frac{3}{2}} - h_2^{\frac{3}{2}})}{h_1 - h_2}$$

∴ actual discharge

$$Q = \tfrac{2}{3}\,C\,L\,\sqrt{2g}\,(h_1^{\frac{3}{2}} - h_2^{\frac{3}{2}}) \text{........15.}$$

It will be seen that if h_2 is Zero, the orifice E C becomes a notch, and the equation 15 reduces to equation 14.

Fig. 28



There is another way of looking at this problem. Treat W C as one notch and W E, another notch. The notch E C is difference of the two, the length L remaining the same in both cases.

Applying equation 14, we get

$$\tfrac{2}{3}\,C\,L\,h_1^{\frac{3}{2}}\sqrt{2g} - \tfrac{2}{3}\,C\,L\,h_2^{\frac{3}{2}}\sqrt{2g}$$

$= \frac{2}{3}\,C\,L\,\sqrt{2g}\,(h_1^{\frac{3}{2}} - h_2^{\frac{3}{2}})$ = discharge through orifice E C. This is similar to equation 15.

As in the case of a notch and for a similar reason, coefficient C is not constant but varies for different heads and areas of orifice, the mean value being 0.62.

Practical examples of rectangular orifices are sluice openings in Barrages, tank Bunds, Locks, etc.

Example I

The discharge from sluices is genereally calculated on the assumption that the velocity at mean depth is the mean velocity. What difference in C. ft. per minute would this make in the calculated discharge from a sluice 4 feet long and 2 feet deep, with a head of 12 feet on the sill, the coefficient bein taken as $\frac{5}{8}$.

True discharge

$$Q_1 = \tfrac{2}{3}\,C\,L\,\sqrt{2g}\,(h_1^{\frac{3}{2}} - h_2^{\frac{3}{2}})$$
$$= \tfrac{2}{3} \times \tfrac{5}{8} \times 4 \times 8\,(12^{\frac{3}{2}} - 10^{\frac{3}{2}})$$
$$= 132.\ 665 \text{ Cusecs.}$$

Velocity at mean depth is

$$= \sqrt{2g \times 11}$$

Approximate discharge

$$Q_2 = C\,A\,\sqrt{2g \times 11}$$
$$= \tfrac{5}{8} \times 8 \times 8\,\sqrt{11}$$
$$= 132.\ 665$$

Difference is 0·046 C. Feet/Sec. or $2\frac{3}{4}$ C. ft. per minute.

32. Triangular Notch.

In the case of triangular notch there is no base to cause contraction, which will be due to sides only. The coefficient of contraction is constant for all heads.

The exact formula for the discharge has been arrived as given below :—

Consider Horizontal filaments at different depths from the water surface which is the base of the triangle.

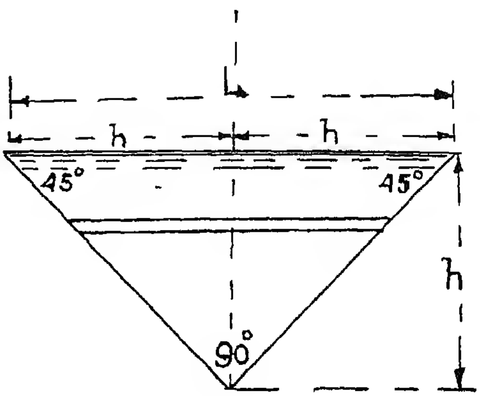
The sum of discharges of all such filaments by Calculus is the discharge of the △ notch.

$Q = \frac{4}{15} C \sqrt{2g} L h^{\frac{3}{2}}$ ....16.

h being the depth of Apex and L the length of base of notch.

It will be seen that the ratio of $\frac{L}{h}$ is constant for all values of h. This is not so in the case of rectangular notch. Here C therefore is more or less constant for all values of h and is equal to 0·617.

Fig. 29



Discharges through similar notches will depend on their linear dimensions, raised to power $\frac{5}{2}$.

Discharge varies as area × Velocity.

Area varies as h^2 and velocity as $h^{\frac{1}{2}}$; therefore Q varies as $h^2 \times h^{\frac{1}{2}}$ or $h^{\frac{5}{2}}$ or $= k\, h^{\frac{5}{2}}$, K being constant, same for all similar notches. This is called Thomson's principle of Geometric similarity.

For a right angled isosceles triangular notch with sharp metal edges, Fig. 29.

$Q = \frac{4}{15} \times \cdot617 \times \sqrt{2g}\, L\, h^{\frac{3}{2}}$: $C = \cdot617$ (UNWIN)

$= \frac{4}{15} \times \cdot617 \times 8 \times 2\, h \times h^{\frac{3}{2}} = 2\cdot632\, h^{\frac{5}{2}}$*........................16A.

Thus knowing the discharge under a head of 1 foot, that under the head of 4 feet will be just 32 times as much.

The writer used this form of notch for gauging small discharges in Simla Hill streams in 1909. For a table of discharges through such a notch see page 37.

Gauging small quantities of water through a Triangular Notch.

The Notch must be right angled notch with thin edges.

H = height above bottom of notch in inches.

* $2\cdot48\, h^{2\cdot48}$ Barnes Hydraulic flow Reviewed.

TABLE OF DISCHARGE, GALLS. PER MINUTE.‡

H	0	·1	·2	·3	·4	·5	·6	·7	·8	·9
1	2·01	2·51	3·12	3·81	4·59	5·45	6·45	7·45	8·59	9·84
2	11·19	12·64	14·20	15·86	17·65	19·55	21·56	23·42	25·95	28·32
3	30·84	33·46	36·23	39·12	42·15	45·33	48·64	52·09	55·67	59·41
4	63·30	65·79	71·50	75·83	80·32	83·74	89·89	94·72	99·85	105·12

5 inches = 110·57

Water dammed; not in motion.

Discharge in gallons $= 1 \cdot 978\ H^2 \sqrt{H}$, or

(Thomson's formula) $= 1 \cdot 91\ H^{2 \cdot 5}$

33. Circular Orifice.

The circle may be divided into a number of vertical strips, which are approximately rectangles. The mean velocity through each rectangle is very nearly the velocity at the centre of gravity of the rectangle. Such centres of gravity lie on the horizontal diameter of the circle. The mean velocity $= \sqrt{2gh}$ nearly, h being the depth of the centre of the circle.

$Q = C\ A\ \sqrt{2gh}$ ....17.

In a similar manner, may be treated the case of any orifice whose form is symmetrical above and below a horizontal axis. When the circumference of the circle touches the water surface, in other words h = radius, then the above formula gives results 4 per cent too much.

34. Submerged Orifice.

This is treated in a preliminary way in article 24. Calling discharge as Q, area of orifice as A, difference of level between the water surfaces as h, then

$Q = C\ A\ \sqrt{2gh}$ ....18.

The proof is as follows: Let B C (Fig. 30) be an elementary fluid thread, the velocity at B being insensibly small. Then we have

At B the head is h^1, the pressure $\pi + wh^1$, the velocity zero.

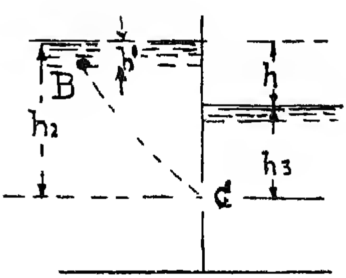
At C the head is h_2, the pressure $\pi + wh_2$, the velocity is v. By Bernouillis theorem

$$\frac{V^2}{2g} + \frac{\pi + w\,h_3}{W} - h_2 = 0 + \frac{\pi + w\,h_1}{W} - h_1 = \text{constant}$$

$$\therefore\ \frac{V^2}{2g} = h_2 - h_3 = h$$

Coefficient C = 0·62. For rectangular orifice, 0·80 for head sluices, 0·90 to 0·95 for bridge openings with curved cut-waters.

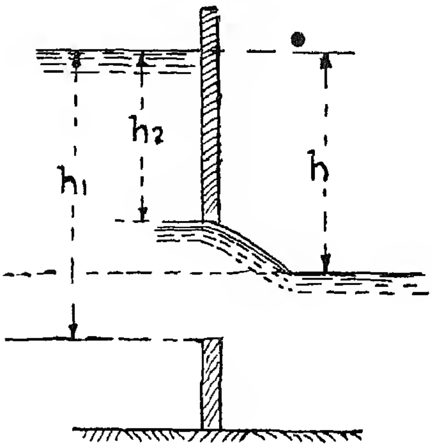
Fig. 30



‡ Molesworth Pocket Book, page 365.

35. Partially Submerged Orifice.

Fig. 31



The discharge may be divided into 2 portions, *viz.* Q_1 taking place through a simple rectangular orifice of depth $(h-h_2)$ and Q_2 through a submerged orifice of depth (h_1-h). The total discharge $Q = Q_1 + Q_2$

$$= C\,L\,\sqrt{2g}\left\{\frac{2}{3}\left(h^{\frac{3}{2}} - h_2^{\frac{3}{2}}\right) + (h_1 - h)\,h^{\frac{1}{2}}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \; 19.$$

$C = 0{\cdot}62$ for Q_1 and for Q_2.

$C = 0{\cdot}66$ for sluices without side walls.

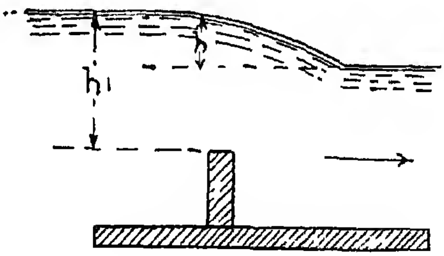
$C = 0{\cdot}70$ for canal locks, and dock gates.

For narrow bridge openings, $0{\cdot}90$:

for wide bridge openings, $0{\cdot}94$.

36. Submerged Notch.

Fig. 32



We may divide the discharge into 2 parts, upper and lower. The former corresponding to an ordinary discharge over a notch and the second to discharge through *a* submerged orifice.

Thus first part of discharge $Q_1 = \frac{2}{3}\,C_1\,L\,H\,\sqrt{2gh}$.

,, second part ,, ,, $Q_2 = C_2\,L\,(h_1 - h)\,\sqrt{2gh}$.

Total discharge $Q = Q_1 + Q_2 = L\,\sqrt{2gh}$

$$\left\{\frac{2}{3}\,C_1\,h + C_2\,(h_1 - h)\right\} \ldots\ldots\ldots\ldots\ldots\ldots 20.$$

For accuracy's sake take the coefficients C_1 and C_2 separately. $C_1 = 0{\cdot}577$ and $C_2 = 0{\cdot}80$ generally. For velocity of approach, h becomes $h + h\alpha$ in the above formula.

The ordinary cases to which these two formulæ are applied in practice, are drowned tank weirs or Barrages; also under bridge openings where the natural water way of the stream has become so constricted as to cause an appreciable amount of heading up of the water on up stream side of the bridge.

When h equals $\frac{h_1}{3}$, standing wave conditions exist. When C is taken equal to C_1 and equal to $C_2 = 0{\cdot}62$ and h taken equal to $\frac{h_1}{3}$, then the down water level has no effect on the discharge.

37. Mouthpieces or Adjutages.

In article 25, discharge from cylindrical mouthpiece is discussed in detail. In article 22, a table is given describing the coefficients of discharge for different kinds of mouthpieces. It will be seen that in the case of divergent conical tube, C is $1{\cdot}46$ and for divergent tube with bell mouth C is $2{\cdot}00$. Thus the actual discharge is more than the theoretical discharge ; without entering into mathematical details, simple explanation is given below.

In article 25 it is explained that there is partial vacuum at "Vena-Coutracta" and the velocity is great. The discharge does not depend on the section of "Vena-Contracta" which may be as small as we please, provided the velocity at "Vena-Contracta" does not become so great as to cause the pressure to fall below zero. Should this occur, the water pressure becomes a tension, continuity of flow would be impossible and the mouthpiece could not run full at the outer end. The maximum theoretical discharge,

$Q =$ (area of the orifice) $A \times \sqrt{2g(h + 34)}$:

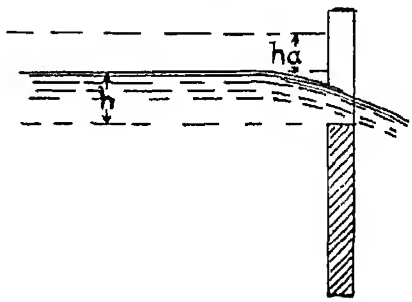
The actual discharge in practice $Q = CA\sqrt{2gh}$,

C being more than unity as given above.

In cylindrical and convergent mouthpieces, there is a sudden contraction at the inner orifice and a subsequent expansion to fill the tube. A loss of energy consequently takes place, which reduces the coefficient of velocity, although the tube may run full at its external end and the coefficient of contraction be equal to unity. The coefficient of discharge in such cases is less than unity as given in table in article 22.

38. Velocity of Approach.

Fig. 33



An important point in connection with all the formulæ given above is that they apply only to the case of discharge from water at rest; thus in each case, the discharge is supposed to take place from some tank or reservoir in which the water is at rest, and maintained by some means at a constant level. A great many practical cases approximate to this and the above formulæ are applicable to them, but there are others, *e.g.* the discharge over a weir across a flowing canal or river in which the water flows up towards the point of discharge with some initial velocity, *e.g.* the average velocity in the canal or river at some distance from the weir. Such a velocity is called a velocity of approach (See article 23), and it should be noticed that velocity of approach is independent of the size and nature of the orifice or notch but depends upon the source which supplies the water for discharge.

This velocity of approach tends to increase the discharge through the orifice or notch.

Fig. 33 shows a rectangular notch. If the velocity of approach is Va, the head required to produce this velocity is $\frac{V_a^2}{2g}$, represented by ha in the figure 33; h represents the original head on the notch. Thus the rectangular notch of depth h becomes a rectangular orifice, the head to bottom being h + ha and head to top being ha. See article 31.

Hence

$$Q = \tfrac{2}{3}\, CL\sqrt{2g}\left\{(h + ha)^{\frac{3}{2}} - ha^{\frac{3}{2}}\right\} \ldots\ldots\ldots\ldots 21.$$

$$C = \cdot 577$$

The construction of a weir across a river causes an increase of water section immediately above the weir; consequently the velocity of approach is less than the natural velocity of the stream.

If A be the natural section, V the natural velocity, let Aa be the section just above weir, Va the velocity.

Then

$$Va\ Aa = VA.$$

The principal cases of discharge to which the above formulæ will have to be applied are (*i*) Tank surplus weirs (*ii*) barrages or anicuts across rivers.

In the above formulæ, square root of different values of h can be found from tables of squares and cubes.

In article 22, a table is given for values of C for orifices or short tubes met with in practice. Some books give values of C for different heads and for different kinds of orifices in elaborate details. These are not copied here.

39. Examples.

(I) An orifice one foot square whose centre is 36 feet below water surface, is found to discharge 36 cubic feet per second. What is the coefficient of contraction, supposing the coefficient of velocity to have its mean value 0.97.

Since the orifice is small, compared with its depth, the formula for discharge is

$Q = CA\ \sqrt{2gb}$: also $C = Cc\ Cv$.

$A =$ one sq. foot.

$\therefore\ 30 = Cc\ \cdot 97\ \sqrt{2 \times 32 \times 36} = Co\ \cdot 97 \times 48$

$\therefore\ Cc = 0{\cdot}64$

(II) A rectangular orifice 4 feet wide and 3 feet high has a water surface 4 feet 6 inches above its lower edge on one side; estimate the discharge and also the amount by which it would be increased if the water had the velocity of approach of 5 feet per second.

See equation 15 of article 31.

Discharge without velocity of approach.

$$Q = \tfrac{2}{3} CL \sqrt{2g}\ (h_1^{\frac{3}{2}} - h_2^{\frac{3}{2}})$$

$$= \tfrac{2}{3} \times \cdot 62 \times 4 \times 8 \left\{ (4{\cdot}5)^{\frac{3}{2}} - (1{\cdot}5)^{\frac{3}{2}} \right\}$$

$= 102$ cusecs nearly

With velocity of approach, 5 feet per second, the head ha, due to velocity of approach,

$= \dfrac{V^2}{2g} = \dfrac{25}{64} = 0{\cdot}4$: See equation 21 of article 38.

$$Q = \tfrac{2}{3}\ CL\ \sqrt{2g} \left\{ (h_1 + ha)^{\frac{3}{2}} - (h_2 + ha)^{\frac{3}{2}} \right\}$$

$= 108$ cusecs

$C = 0{\cdot}62$
$L = 4$
$h_1 = 4{\cdot}5$
$h_2 = 1{\cdot}5$

$\therefore$ increase due to velocity of approach

$= 108 - 102 = 6$ cusecs.

(III) Water enters the condenser of a steam engine at the sea-level from a reservoir whose water surface is 10 feet above the injection orifice. The pressure in the condenser is 3 lbs. per square inch. Find the theoretical velocity of flow into the condenser.

The atmospheric pressure in the reservoir is 14·7 lbs. per square inch. The resultant pressure is thus $14{\cdot}7 - 3 = 11{\cdot}7$ lbs. per □″ or $11{\cdot}7 \times 12 \times 12 = 1685$ lbs. per square foot. This is equivalent to a head of $\dfrac{1685}{62{\cdot}4} = 27$ feet. The total effective head is therefore

$27 + 10 = 37$ feet.

$V = \sqrt{2gh} = 8\sqrt{37} = 48{\cdot}7$ feet.

(IV) The discharge from sluices is generally calculated on the assumption that velocity at mean depth is mean velocity. What difference in cubic feet per minute would this make in the calculated discharge from a sluice 4 feet long and 2 feet deep, with a head 12 feet on the sill, the coefficient being $\frac{5}{8}$.

The mean depth $= 12 - 1 = 11$ feet only.

Mean velocity $= \sqrt{2gh} = \sqrt{2g \times 11}$: approximate discharge $= CA \sqrt{2g \times 11} = \frac{5}{8} \times 4 \times 2 \times 8 \times \sqrt{11}$
$= 132 \cdot 665$ cusecs.

True discharge $Q = \frac{2}{3}$ CL $\sqrt{2g}\ (h_1^{\frac{3}{2}} - h_2^{\frac{3}{2}})$

See equation 15 of article 31.

$h_1 = 12$ feet and $h_2 = 10$ feet

$$Q = \frac{2}{3} \times \frac{5}{8} \times 4 \times 8 \times (12^{\frac{3}{2}} - 10^{\frac{3}{2}})$$
$$= 132 \cdot 619.$$

Difference $= 132 \cdot 665 - 132 \cdot 619 = 0 \cdot 046$ cusecs.

(V) A culvert 3 feet long, consisting of a semicircular arch of one foot radius resting on a level floor, has to pass a discharge of 9 c.ft. per second. There is a free fall downstream. What will be water level upstream.

This is a case of a semicylindrical tube, diameter 2 feet : length 3 feet : cross section = a semicircle of 2 feet diameter : area $= 1 \cdot 57$ sq. ft.

From table article 22

$C = \cdot 80 =$ Coefficient of discharge.

Discharge $Q = 9 = \cdot 80 \sqrt{2gh} \times 1 \cdot 57$

Or $h = \cdot 80$ feet = height from the centre of gravity of the aperture to free water surface. Now the centre of gravity is $\cdot 58$ feet from the crown of the arch downwards

$\therefore\ 0 \cdot 80 - 0 \cdot 58 = \cdot 22$ feet : This is the height of free water surface above the crown of arch.

(VI) If the discharge takes place through a triangular notch in a waste board placed on a dam, the two sides of the notch being equally inclined and meeting at a right angle, find the coefficient of discharge C when discharge $Q = \frac{1}{3}\sqrt{h^5}$, where h is the height in inches of still water above bottom of notch and Q is the discharge in cubic feet per minute.

The approximate discharge in cusecs $= CA\sqrt{2gH}$, where H is head in feet to centre of gravity of water section.

In this case $H = \dfrac{h}{3} \times \dfrac{1}{12} = \dfrac{h}{36}$ feet.

Since the traingle is an issoceles one, the base $L = 2h$ (inches) : Area $= (2h \times h) \times \dfrac{1}{144} \times \frac{1}{2} =$ $\dfrac{h^2}{144}$ sq. ft.

Approximate discharge per minute

$$= \left\{ C \times \left(\frac{h^2}{144}\right) \times \sqrt{\frac{2gh}{36}} \right\} \times 60.$$
$$= \frac{5}{8}\, C \sqrt{\frac{h^5}{3}}$$

By Question, $Q = \frac{1}{3}\sqrt{h^5}$

$$\therefore\ \frac{1}{3}\sqrt{h^5} = \frac{5}{3}\ C \sqrt{\frac{h^5}{3}}$$

$\therefore\ 1 = \frac{5}{3} C \qquad \therefore\ C = \frac{3}{5} = 0 \cdot 60$

(VII) What will be the discharge per minute through a rectangular notch, in a thin plate 6 feet wide, head 8 inches, velocity of approach 2 miles per hour ?

See article 30 and 38

Velocity of approach $Va = \dfrac{2 \times 5280}{60 \times 60} = 2 \cdot 93$ feet per second.

Head due to velocity of approach $= h\alpha = \dfrac{V\alpha^2}{2g} = \dfrac{(2\cdot93)^2}{64} = 0\cdot13$ feet. The total head on notch $= h + h\alpha$

$= \dfrac{8}{12} + 0\cdot13 = 0\cdot80$ feet.

Length of notch L = 6 feet.

$$\text{Discharge } Q = \tfrac{2}{3}\, CL\, \sqrt{2g} \left\{ (h + h\alpha)^{\frac{3}{2}} - h\alpha^{\frac{3}{2}} \right\}$$

$$= \tfrac{2}{3} \times \cdot62 \times 6 \times 8 \left(0\cdot80^{\frac{3}{2}} - 0\cdot13^{\frac{3}{2}} \right) = 20 \times 0.667 = \frac{40}{3}$$

$\therefore$ discharge per minute $= \dfrac{40 \times 60}{3} = 800$ cusecs.

(VIII) Find the discharge from a circular Bellmouthed tube, 1 foot in diameter, situated in the middle of the end of a horizontal trough of rectangular section, 2 feet wide and 2 feet deep.

See articles 21 and 22

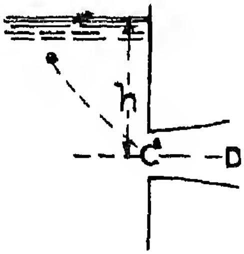
The head h is one foot from the centre of Bellmouth to the top of free surface water level.

Area A $= 0\cdot785$ sq. feet. C $= 0\cdot97$

Discharge $Q = CA\sqrt{2gh} = 0\cdot97 \times \cdot785 \times 8 \times 1 = 6\cdot08$ cusecs.

(IX) The external area of a divergent concoidal mouthpiece is 4 sq. inches. Find the theoretical area of the smallest cross section, which will allow the mouthpiece to run full. (*i*) When the head of water is 2 feet, (*ii*) when it is 15 feet.

Fig. 34



(*i*) At point D, the velocity is

$\sqrt{2gh} = \sqrt{2g \times 2}$. The area is 4 sq. inches.

At point C, the velocity is $\sqrt{2g\,(2 + 34)}$.

The area at C is say A sq. inches.

The discharges at C & D are equal.

$\therefore\ A\sqrt{2g \times 36} = 4\sqrt{2g \times 2}$ $\quad \therefore\ A = 0\cdot94$ sq : inches

(*ii*) $A\sqrt{2g \times 49} = 4\sqrt{2g \times 15}$

$\therefore\ A = 2\cdot21$ square inches.

(X) A circular orifice 1 square inch in area is made in the vertical side of a large tank. If the jet fall vertically through $1\frac{2}{3}$ feet while moving horizontally through 5 feet, at the same time discharging 16 gallons per minute, determine the horizontal force on the tank.

Answer : 1·286 lbs.

(XI) Find the time required to empty a swimming bath through a flat grating in the bottom of the deep end.

Depth of water at deep end	=	6 feet
,, ,, ,, shallow end	=	3 ,,
Length of bath	=	80 ,,
Breadth	=	30 ,,

Area of grating = 2 square feet : coefficient of discharge 65.

Answer : 596 seconds.

CHAPTER VIII.

WEIRS AND PRACTICAL CASES OF DISCHARGE FROM LARGE ORIFICES AND NOTCHES.

40. *Weirs.*—In the preceding chapter we have dealt with discharges from large orifices and notches. We now proceed to weirs, etc.

A weir is essentially an irregularity in the stream bed, over which the water falls in a sheet of certain depth. The term weir refers to the whole constructional apparatus used to produce the definite sheet.

In a sharp edged weir, Nappe or stream springs clearly of the sides provided there is a clear margin all round the weir.

In the case of discharge through a thin plate, the coefficient is certainly known with some precision, but, except in the cases of gauging small streams, discharges take place in practice through masonry works, whose varying details of construction render it impossible to assign coefficients which will always suit each type of work.

Unless the contrary is stated, it will be assumed that all weirs have vertical side walls, such forming in practice the vast majority, and the contraction is complete.

The practical formula for discharge over a sharp-crested weir with complete contraction and velocity of approach is

$$Q = \frac{2}{3}\ C\ L\ h^{\frac{3}{2}}\ \sqrt{2g} \qquad \text{......22.}$$

Where L is the length of the crest, h the head on the crest and C is a coefficient of discharge whose value for weirs in thin walls averages about 0·62 and for others, varies according to the Form of weirs, and value of h, irrespective of the value of additional head due to velocity of approach.

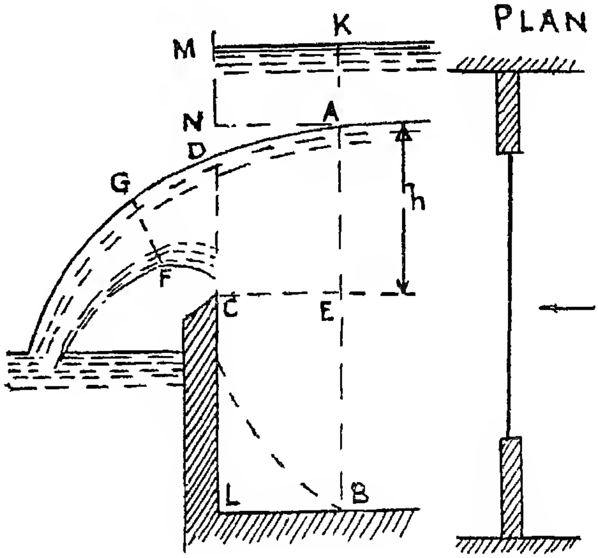
Description of weir.	Value of C.
Broad crested or flat topped weir	0·577
Weirs with narrow crests	0·623
Weirs over-fall where L = full width of channel (end contractions suppressed)	0·666

For broad crested weirs $Q = 3 \cdot 1\ L\ h^{\frac{3}{2}}$.

For short crested weirs, see paper No. 170 of P. E. Congress by C. C. Inglis, S. E. P. W. D., Bombay and also see Bombay P. W. D. Technical paper No. 44 "The Dissipation of Energy below falls" by C. C. Inglis.

The case is similiar to a rectangular Notch as described in article 30 of chapter VII. At a weir, see fig. 35 the water surface always begins to fall at a point A, situated a short distance upstream of the weir. Hence whatever the crest and end contractions may be, there is always surface contraction. The angular spaces between the wall and bed and sides of channel are occupied by eddies. The fall in the surface begins where the eddies begin. From this point, the section of the stream proper or forward moving water diminishes, its velocity and momentum increase and the increased surface fall is necessary to give the necessary momentum.

Fig 35



When the notch boundary and walls of approach channel coincide at a point, the contraction is completely suppressed at that point.

The effect of partial suppression of contraction is to increase the discharge, the more the suppression the more the discharge.

The effect of the side contraction is to decrease the discharge by an amount

$$\frac{4}{3}\ \frac{C}{10}\ \sqrt{2g \times H^{2\cdot 5}} \ \ldots\ldots\ldots\ldots \quad \text{(Francis)}.$$

The head 'h' should always be measured above the point where the flow of water is uniform.

Effective head H = h + ha (the head due to velocity of approach).

For a given weir in a thin wall, c decreases as h increases, the effect of end contractions increasing with 'h.' It has been stated that if sides are given a slope of $\frac{1}{4}$ to 1, c is constant for all heads, the sloping sides having the effect of lengthening the weir as h increases. In the case of flat topped weirs with narrow crests, the discharge depends upon (*i*) ratio of depth of water over crest to the width of crest top, (*ii*) upstream face batter, (*iii*) down stream face batter, (*iv*) height of crest above upstream bed-level, (*a*) if the upstream face has a batter of 1 in 3 and (*b*) the down stream face a batter of 1 in 8, the discharge increases by 18 per cent for (*a*) and 23 per cent for (*a*) + (*b*).

Professor Gibson in his book *Hydraulics*, page 160, Third edition, says "Except with a freely discharging *Nappe* or water sheet, the state of affairs is very unstable. Any pulsation in the stream flow, any floating body piercing the *Nappe* and allowing admission of air, or even a sudden gust of wind blowing the *Nappe* into contact with the weir face, may totally alter the conditions of flow, and no attempt should be made to use a weir, except when discharging freely, as a measuring device."

41. *Velocity of approach.*—If there is velocity of approach, the formula is

$$Q = \tfrac{2}{3}\ C\ L\ \sqrt{2g}\ \ (h + ha)^{\frac{3}{2}} \ \ldots\ldots\ldots\ldots 23.$$

V = mean velocity of approach

$$ha = (\text{head due to velocity } v) = \frac{v^2}{2g}$$

h + ha = H = Effective height or head.

For flat topped weirs $C = 0{\cdot}577$.

Weirs with narrow crests $C = {\cdot}623$

Over-fall weirs where L is the full width of the channel (*i.e.*, end contractions suppressed) $C = {\cdot}666$.

It is some times assumed that the effect of energy due to velocity of approach is the same as that of raising the water level by a height AK (Fig. 35), equal to $\frac{v^2}{2g}$, the discharge is the same as that through an orifice with heads KA & KE. The discharge is the difference between the discharges of two weirs (i) crest c, head KE and (ii) crest at A and head KA.

$$Q = \tfrac{2}{3} C L \sqrt{2g} \left\{ (h + ha)^{\frac{3}{2}} - ha^{\frac{3}{2}} \right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad 24.$$

This is similar to equation 15 in article 31.

The velocity of approach can be measured when the water is flowing over the weir. But when the weir is not even constructed and is being designed in office, then calculate the velocity of the ' Approach channel ' approximately, the bed slope and cross section being given, by trial and error from the equation 23, fair values of v, h & Q can be ascertained.

Francis formula, adopted by French officials, for velocity of approach is as under :—

Effective or still water head $H = (h + ha)^{\frac{3}{2}} - ha^{\frac{3}{2}}$

H = effective depth

h = observed depth of water over crest.

v = mean velocity of approach.

$$ha = \frac{v^2}{2g}$$

The above relationship does not strictly hold good in the case of flow of water over weirs with flat narrow crests on account of the fact that in the approach channel the distribution of velocities is not at all uniform in any vertical, the velocity at the top being much higher than that at the bottom. Bazin and others have therefore advised that the effective additional head over such weirs due to velocity of approach V_a is $1{\cdot}5 \frac{V^2}{2g}$, depending on the type of the crest and conditions of flow in any particular case.

Francis* gives the following equation :—

For discharge over weirs with end contractions,

$$Q = 3{\cdot}33\,(L - {\cdot}1\,n\,h)\,h^{\frac{3}{2}}.$$

L = Length of weir.

h = Observed head.

n = Number of side contractions.

= 2 in an ordinary weir.

= 4 in a weir with a sharp edged pier in its midst.

H = corrected head = $h + ha$.

ha = head due to velocity of approach = $\frac{V^2}{2g}$.

If there be a velocity of approach, substitute H for h in the above formula.

* An American mathematician deduced this formula experimentally, which is quite rational. $Q = 3{\cdot}33\,H^{\frac{3}{2}}$ if $L = 1$. $\log Q = \log 3{\cdot}33 + \frac{3}{2}\log H$, a straight Line Law.

In the case of a weir having the same length as the width of approach channel n is zero:

Then $Q = 3\cdot33\ L\ H$.

42. *Various kinds of weirs and their coefficients.*—In the case of weirs having an upstream slope, the discharge increases according to some higher power of the head than $\frac{3}{2}$. The power may attain a value as high as 1·75, although it usually lies between 1·5 and 1·65 (Prof. Gibson).

Discharge is increased by flattening the up stream face batter and also by flattening the down stream face batter.

Discharge is reduced by increasing the width of the crest in proportion of $B^{\frac{1}{6}}$, B being the width of the crest.

For a crest having up stream face batter 1-3 and down stream face batter 1-8,

$$Q = 3\cdot61 \left(\frac{H}{B}\right)^{\frac{1}{6}} H^{\frac{3}{2}}\ L.$$

H = Effective depth of water

B = Breadth of crest

L = Length of weir.

If in the above the down stream face is vertical,

$$Q = 3\cdot324 \left(\frac{H}{B}\right)^{\frac{1}{6}} H^{\frac{3}{2}}\ L.$$

For a broad crested weir

$$Q = 3\cdot00\ L\ H^{\frac{3}{2}}$$

See paper No. 141 : P. E. C. (1930) for energy losses over long crested weirs by Bedford and Montagu.

43. *Surplus or waste weirs.*—The writer visited in 1902 an artificial lake built in the last quarter of 19th Century. This lake is situated about 10 miles from Poona in a hilly country at Khadakwasla.†

An extensive piece of more or less flat land, surrounded by hills forms the basin of the lake. The surrounding hills form the catchment area from which rain water flows towards the lake. A long masonry dam about 75 ft. high has been built to form the lake.

A part of the dam, at one end is the waste weir. A natural torrent with rocky bed forms the escape channel. A canal with head works at the other end has been built along the hill side to use the lake water for irrigation purposes.

The inflow into the lake is $Q = C\ M^{\frac{2}{3}}$, where M is the area of the catchment basin in square miles and c is a local coefficient ranging from 350 to 650, depending upon the nature of the soil and slope of the ground in the catchment area, and Q in cusecs. The above empirical formula is due to Ryves. Dicken's formula $Q = C\ M^{\frac{3}{4}}$ is also some times employed, c being a coefficient, whose value depends upon local conditions.

The top of the dam is generally 4 ft. above the maximum water level in such a lake. The depth or head on weir is generally 2 to 4 feet, the crest thus being 4 feet below the maximum level in the lake.

† Sir M. Visvesvaraya, an eminent Engineer, now in Bombay (1948), was then Ex. Engineer in charge.

The top or crest of weir is 3 ft. broad or a little more. Assuming that the lake is full when the rainfall commences, then this is the maximum discharge which the weir should pass with the given head The discharge provided for is proportional to the intensity of rainfall, that is to say the maximum rainfall per hour. On the top of the weir at Khadakwasla§, iron sluices are built which open automatically, when the flood level reaches a certain mark; The weir has a clear fall on the outside vertical face.

The discharge over such a weir is that through a rectangular notch (see equation 14), *viz,* $Q = \frac{2}{3}\, cLh\, \sqrt{2gh}$ ....25.

h being the head over crest, measured to the surface of still water. The Value of C, the coefficient, varies with the head, with the length and thickness of weir crest and with the depth of water in front of the weir. C is ·5777 as proposed by Prof. Unwin. Some times sluice syphons or spillway syphons are used to allow the surplus water to escape, see article 76.

Example 1. What length of escape weir should a storage tank have for each square mile of a catchment basin in order to carry off a rainfall of one inch per hour, of which 60 per cent reaches the tank, it being assumed that the tank is full, and the supply from each square mile is uniform and the height of still water above the crest of the weir is 4 ft.

$$\text{Discharge in cusecs, } Q = 5280^2 \times \frac{1}{12} \times \left(\frac{1}{60^2} \times \right) \frac{60}{100} = 387{\cdot}2$$

Now $Q = \frac{2}{3}\, c\, L\, h\, \sqrt{2gh}.$ See equation No. 25.

$387{\cdot}20 = \frac{2}{3} \times {\cdot}577 \times L \times 4 \times 8 \times 2$

or $15{\cdot}7 = L.$

Thus for 1 sq. mile of catchment area, the length of the weir should be 15·7 feet.

The above results also agree with the figures on Nomogram No. 1.

If it is required to find time in seconds to reduce the water level from height h_2 to h_1 above the sill of the weir, of given length, the following formula (Francis) to be used.

$$T = \frac{2\,A}{3{\cdot}33\,(L - .1nh)} \left(\frac{1}{\sqrt{h_2}} - \frac{1}{\sqrt{h_1}} \right)$$

A = area in sq. feet of the lake between contours at h_2 and h_1.

h = mean of h_2 and h_1

n = Number of contractions at sides.

Example 2. A weir in a thin wall is 25 feet long and 3 ft. high. The head of water over crest is 1 foot. The channel of approach is 30 feet wide, find the discharge Q.

The crest contraction and end contractions are complete. The Bed slope of approach channel is not known. The velocity of approach is also not known. First find the discharge approximately, leaving out the velocity of approach.

$Q = \frac{2}{3}\, c\, L\, h\, \sqrt{2gh} = \frac{2}{3} \times {\cdot}577 \times 25 \times 1 \times 8$

$= 76{\cdot}93$ cusecs.

The sectional area of approach channel = 30 × (3 + 1) = 120 sq. ft.

Discharge $Q = 76{\cdot}93 = 120 \times V_a$, V_a being the velocity in approach channel approx.

$\therefore\ V_a = 0{\cdot}64$ feet per second.

The head h_a due to this velocity $= \frac{V_a^2}{2g} = {\cdot}0064$ feet.

$\therefore$ the effective head $= 1 + {\cdot}0064 = 1{\cdot}0064$ feet.

The correct discharge $Q = \frac{2}{3}\, c\, L\, h\, \sqrt{2gh}$

$= \frac{2}{3}\, c\, L\, \sqrt{2g} \times h^{\frac{3}{2}}$

$= \frac{2}{3} \times {\cdot}577 \times 25 \times 8 \times 1{\cdot}01 = 77{\cdot}69$ cusecs.

This agrees with the readings on Nomogram No. 1, if a straight edge is applied.

§ An up-to-date Hydro-dynamical Research Laboratory is located here and conducted by P. W. D. Engineer.

Example. 3. A tank has a drainage area of 45 square miles and a weir 250 feet long. It is required to find the height of the bund above the escape crest in order to give a margin of 6 feet above maximum flood level. Formula for discharge from the catchment basin $Q = 500\ M^{\frac{2}{3}}$

Here $Q = 500 \times 45^{\frac{2}{3}}$ = actual discharge.

Theoretical discharge over escape weir $Q = \frac{2}{3}\ CL \sqrt{2g}\ h^{\frac{3}{2}}$

$\therefore 500 \times 45^{\frac{2}{3}} = \frac{2}{3} \times \cdot 577 \times 250 \times 8 \times h^{\frac{3}{2}}$

$$\text{or } h^{\frac{3}{2}} = \frac{2 \times 45^{\frac{2}{3}}}{3 \cdot 07}$$

whence h = 4 feet.

$\therefore$ height of bund or Dam = 6 + 4 = 10 feet.

Side Weir :—

In New Delhi Sewerage scheme, an overflow on the left side of 84 inches diameter sewer was built. The sill level of this side overflow weir was kept at the centre of the Sewer which carried Sewage during dry season upto the sill of the weir : During rains, the excess passed over the weir into a natural stormwater drain leading to the river Jamuna.

$L = 29 \cdot 1\ w^{1 \cdot 4}\ h^{\cdot 513}$ feet

$Q = 1 \cdot 674\ w\ L^{\cdot 72} \times h^{1 \cdot 645}$ cusecs

L = Length of the sill of the weir.

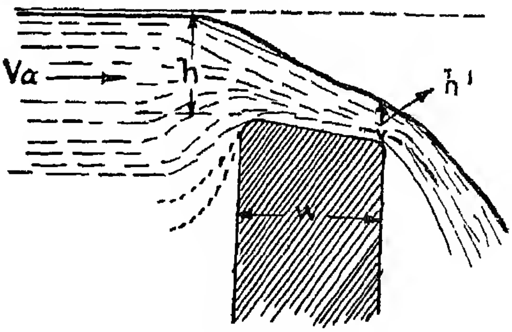
W = width of the main channel (84 inches sewer in this case).

h = head over the sill.

(" Proceedings Inst. C E." 1923).

44. *Weirs with broad sloping crest.* —The form of such a weir is shown in fig. 36. The crest is sloping and broad the edges are slightly curved to suppress contraction. If the width of weir is great, the coefficient of discharge is a function of the head, the length of the crest, the roughness of the crest surface, the coefficient of viscosity and the temperature. Professor Unwin takes C as ·577. For maximum discharge h^1 should be 2/3 of h : $V = \sqrt{2g\,(h-h^1)}$. For discharge formula see article 43.

Fig. 36.



$Q = \frac{2}{3}\ C\ L\ h\ \sqrt{2gh}$: see equation 14 & 25. C = ·577. If the width of the crest exceeds 3 feet, then

$$Q = 3 \cdot 00\ L\ H^{\frac{3}{2}} \qquad \ldots\ldots 26.$$

H, being effective depth and greater than 1·5w.

$H = h + \dfrac{Va^2}{2g}$, see P. E. C. volume 1919.

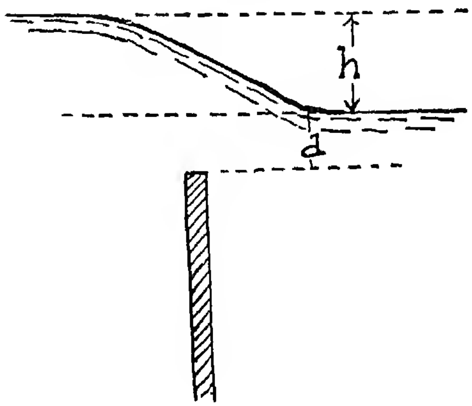
(See " weir experiments, coefficients and formulas " by R. E. Horton).

45. *Drowned Tank Weirs.*—If the crest is low and the surplus channel confined, the tail water may sometimes rise above the level of the crest. This is the case of a submerged notch, see equation 20 of article 36. Velocity of approach is neglected. h can be found from known flood discharge. Formation of standing wave on lower side also complicates the problem.

A paper named " flow of water over masonry weirs and notches " was submitted to the Institution of Engineers (India) by Mr. M. L. Garga, Executive Engineer in October 1935. The results of his experiment and investigations are noted here:—

Kheri branch of Sarda canal has a fall with broad crested weir. It was observed that at a maximum submergence of about 41 per cent, it was found to be behaving as a perfectly free overfall crest. This is in keeping with the fact that the coefficient of discharge through weirs is assumed to remain unaltered as long as the submergence does not exceed 67 per cent.

Fig. 37



Example 3. A weir has a head of 4 feet of water on its crest and the tail water rises 3 feet above the crest. Find the discharge per second for each length of 15·7 feet. See formula 20 of article 36.

Whose $h_1 - h$ is termed d here

$Q_1 = \frac{2}{3} C_1 L h \sqrt{2gh}$:

$Q_2 = C_2 Ld \sqrt{2gh}$: $C_1 = \cdot 577$

$C_2 = \cdot 80$

d = 3 feet

h + d = 4 feet

h = 1 foot

$$\therefore Q = Q_1 + Q_2 = 15 \cdot 7 \times 8 \left\{ \left(\tfrac{2}{3} \times \cdot 577 \right) + (\cdot 8 \times 3) \right\} = 350 \text{ cusecs.}$$

46. *Trapezoidal weir.*—Discharge

$$Q = \tfrac{2}{3} C \sqrt{2g}\, h^{\frac{3}{2}} \left\{ L + (\cdot 8 \operatorname{Cot} \Theta\, h) \right\} \quad \ldots\ldots 27.$$

The weir has a free fall.

The value of C = 0·577 for notches in thin walls (unwin)

= 0·75 for large notches on canals (Sir John Benton)

= 0·78 for canal notches (A. G. Reid)

= 0·79 for distributory notches (A. G. Reid)

C varies with value of h.

Fig. 38

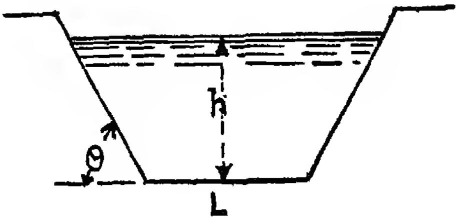
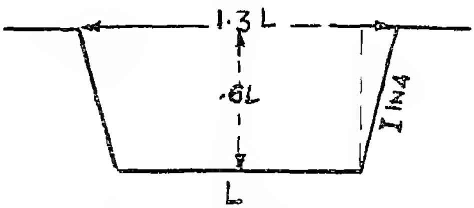


Fig. 39



47. *Cippoletti weir.*—For large discharges this weir may be used with a $\frac{1}{32}''$ sill cut away at 45°, h = height of water over sill. h not to be more than 1/3rd the length of sill.

$Q = 3 \cdot 367\ L\ h^{\frac{3}{2}}$ (Molesworth) : L = Length of sill.

The velocity of flow or approach should be nill.

There should be free overfall.

48. *Gauging weir.*—When it is desired to accurately measure the discharge of a stream, a dam made of piles and planks is formed inside the stream and puddled on the inside to prevent leakage. A notch, generally rectangular, of suitable size to carry the discharge, is made in the weir and is protected by a metal plate, 1/10 of an inch thick, so as to secure accuracy of form and permanent sharpness of edge. The air must have free access behind the sheet of falling water.

This is a case of discharge through a rectangular notch with velocity of approach.

If the section of the jet does not exceed 1/5 of water section above the weir, the velocity of approach may be neglected. The head is measured on a scale placed on a pile, the zero of the scale being accurately level with the crest of the notch. Sometimes the scale is placed in a well, built a little away from the weir upstream side, so as to keep the scale safe from eddies, etc.

Sometimes a hook gauge is employed. This is available in the market from firms dealing in Hydraulic appliances, etc.

If the head is variable, the scale should be read at intervals of certain number of hours and the discharge of any interval be calculated by the mean of the heads at the beginning and end of that interval. The discharge is estimated by equation No. 22 :

$Q = \frac{2}{3}\ CLh\ \sqrt{2gh},$ where $C = 0 \cdot 62$ for ordinary heads. If the water contains silt, then a triangular notch or cippoletti weir is recommended.

Example 4.—A rectangular notch is 1·5 ft. wide and the head to still water is 0·64 feet. Find the discharge per second.

$Q = \frac{2}{3} \times 0 \cdot 62 \times 1 \cdot 5 \times 0 \cdot 64 \times 8 \times \cdot 8 = 2 \cdot 54$ c. ft. per second.

49. *Anicut or Raised weir or Barrage.*—This is a masonry dam built across a river so as to raise the water surface to a sufficient height in the dry season to admit of the water being carried by gravitation to places where it otherwise could not reach. Fine example of this class of work can be seen (*i*) at Okla on Jumna River at Delhi, (*ii*) at Rasul on Jhelum River in Punjab, (*iii*) at Ferozepur on Sutlej, Head works of Upper Bari Doab canal, Head works of lower Bari Doab canal, Head works of Upper Chenab canal, all in Punjab, (*iv*) finally at Sukkur in Sindh across the river Indus.

The supply of water required is taken off by canals on one or both sides, just above the raised weir, the opening from the river being provided with a masonry head-sluice to regulate the admission of water. If the full supply level is to be maintained in the canal when little water comes down the river, the crest of the raised weir should be slightly above the proposed full supply level in the canal. All surplus water passes over the raised weir. In ordinary seasons the surplus may discharge with a free overfall, but in floods the tail water may rise above the crest of the weir, and the water on upper side becomes heaped up until there is a sufficient head to drive the river discharge through the contracted section. In both cases the velocity of approach must be taken into account.

In some cases, boards or planks called flash boards are put up on the crest of the weir to regulate the full supply level in the river and in the canal during dry season. During floods the wooden boards are removed and the depth of flood water over the crest maintains the full supply level in the off-take channels above the weir which are also well controlled by Head regulators. The raised weir across Jhelum River at Srinagar in Kashmir is a fine example of the skill of Indian Engineers. There is also a lock to allow passage of cargo boats to pass down from the higher part of the river to lower part and *vice versa*.

In some cases, masonry structures are erected on the top of the raised weir, wherein openings fitted with sluice gates are provided. The openings are arched over and carry a foot bridge or road bridge. Sometimes the sluice doors are operated by a crane or an electric motor. The object of this arrangement is to regulate the full supply level or flood level in the river as well as in the canals taking off from the river. The finest example is at Sukkur in Sindh across the river Indus. The reader is advised to visit as many barrages in India as he can.

50. *Raised weir with clear overfall*—This is the case of free discharge from a rectangular notch, with velocity of approach. See article 38, Formula 21.

$$Q = \tfrac{2}{3}\ CL\ \sqrt{2g}\left\{ (h + ha)^{\frac{3}{2}} - ha^{\frac{3}{2}} \right\} \quad \ldots\ldots 29.$$

C = ·577

ha = head due to velocity of approach.

If the value of h is known, then the discharge of river can be estimated. Just above the weir, there is increase in the water section = Aa, and the velocity of approach Va is less than the normal velocity V of the stream water section far above the weir, but the discharge of the stream remaining the same, V A = Va Aa.

Example 5.—A river 200 feet wide and flowing 5 feet deep with a mean velocity of 4 feet per second passes over an anicut or raised weir, whose height is 8 feet above bed, with a clear overfall. Estimate the depth of water on the crest.

Q = (200 × 5) s. ft. × 4 = 4,000 c. ft. per sec. The increased water section above the anciut is unknown. Say it is 200 × 8 = 1,600 s. ft. approx.

The approximate velocity of approach is $\frac{4000}{1600} = 2 \cdot 5$ ft. per sec.

∴ the head due to velocity of approach is $ha = \frac{(2\cdot5)^2}{2g} = 0\cdot1.$

Inserting these values in equation in the article 50 above,

$$4{,}000 = \tfrac{2}{3} \times \cdot577 \times 200 \times \sqrt{2g} \times \left\{ (h + \cdot1)^{\frac{3}{2}} - \cdot1^{\frac{3}{2}} \right\}$$

h = 3·49 feet approximate.

If we assume that there has been no silting, the value of h may be corrected as follows:—

The velocity of approach is:—

200 × (8 + 3·49) × Va = 200 × 5 × 4.

Va = 1·8 ft. per sec.

∴ ha = ·05

substituting the value of ha in the above equation, we get h = 3·43 feet.

Example 6.—A river is 50 feet wide and has a maximum discharge of 600 cubic feet per second. A weir with rounded crest (C = ·80) is to be built in the river so as to raise the flood level by one foot.

What must be the height of the crest above the bed. Velocity of approach being neglected.

$Q = \tfrac{2}{3}\ C\ L\ \sqrt{2g}\ h^{\frac{3}{2}}$: h = the depth of water over crest.

$$600 = \tfrac{2}{3} \times \cdot80 \times 50 \times 8 \times h^{\frac{3}{2}}$$

$\therefore h^{\frac{3}{2}} = 2\cdot81$

h = 2 ft. : (3 + 1) — 2 = 2 height of the crest over bed.

51. *Drowned anicut.*—This is the case of a submerged rectangular notch with velocity of approach.

See article 38, equation 21.

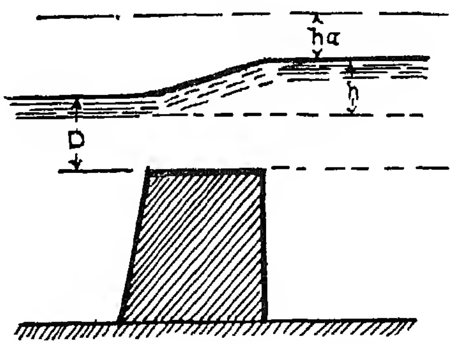
Discharge through h (Notch portion)

$$Q_1 = \tfrac{2}{3} C_1 L \sqrt{2g} \left\{ (h + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}} \right\}.$$

Discharge through D (sluice portion)

$$Q_2 = C_2 L D \sqrt{2g(h + h_a)} :—$$

Fig. 40



$$\left. \begin{array}{l} C_1 = \cdot 577 \\ C_2 = \cdot 80 \end{array} \right\} \text{approximate.}$$

$$Q = Q_1 + Q_2 = L \left[\tfrac{2}{3} \times \cdot 577 \times 8 \left\{ (h + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}} \right\} + \cdot 8 \times d \times 8 (h + h_a)^{\frac{1}{2}} \right]$$

$$= L \left[3 \cdot 07 \left\{ (h + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}} \right\} + 6 \cdot 4 d (h + h_a)^{\frac{1}{2}} \right] \quad \ldots\ldots\ldots\ldots 30.$$

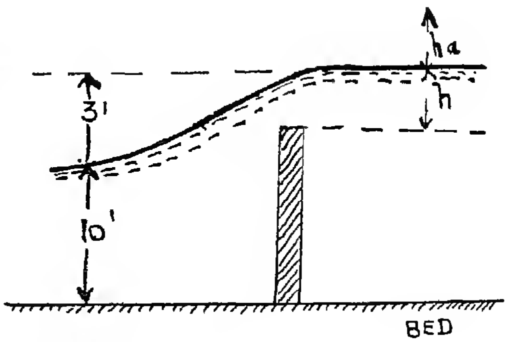
If the anicut were absent, h is nearly zero, C is then unity. If there were a clear overfall, d is nearly zero, C is $\cdot 577$. Thus the average coefficient for $Q_1 + Q_2$ varies between 1 and $\cdot 577$, increasing as the ratio $\frac{h}{d}$ diminishes. This conclusion indicates the imperfections of the formula ; nevertheless it is believed that the above equation will give good results in ordinary cases.

Example 7.—A river 100 feet wide 10 feet deep has a mean velocity of 4 feet per second ; find the height of anicut to raise the water level in the river by 3 feet.

Here we have a velocity of 4 feet per second.

Discharge $Q = 100 \times 10 \times 4 = 4000$ cusecs.

Fig. $\frac{40}{1}$



It is not known whether the anicut will have a clear overfall or not. This can be settled if the height of the anicut is known.

Assume the anicut has a clear overfall see article 50.

$$\text{Then } Q = \tfrac{2}{3} CL \sqrt{2g} \left\{ (h + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}} \right\}$$

Where h is the required depth on crest of weir $\left\{ \begin{array}{l} C = \cdot 577 \\ h_a = \text{head due to velocity of approach.} \end{array} \right.$

The water level in the river higher up the anicut is to be raised by 3 feet: Total depth of water 13 feet. Velocity of approach will be

$$V_a \times 100 \times 13 = 4{,}000 \text{ cusecs}$$

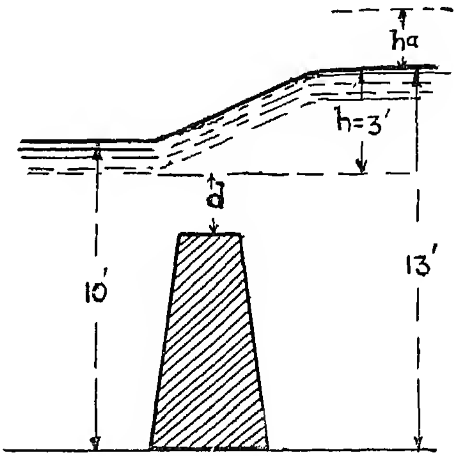
$$\therefore V_a = 3 \text{ feet approximate}$$

$$h_a = \frac{V_a^2}{2g} = \frac{9}{64} = \cdot 14 \text{ feet.}$$

$$\therefore Q = \tfrac{2}{3} \times \cdot 577 \times 100 \times 8 \left\{ (h + \cdot 14)^{\frac{3}{2}} - (\cdot 14)^{\frac{3}{2}} \right\} = 4{,}000 \text{ Cusecs.}$$

$$\therefore h = 5 \cdot 4 \text{ approximately.}$$

Fig. 40 A



Hence $10 + 5 \cdot 4 = 15 \cdot 4$ ft. of water in the river against the given maximum depth of 13 ft.

Therefore the crest of the weir should be lowered further. It must be a drowned weir. We must use the formula for drowned weir, equation 30.

$$Q = Q_1 + Q_2 = L \left[3 \cdot 07 \left\{ (h + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}} \right\} + 6 \cdot 4 d (h + h_a)^{\frac{1}{2}} \right]$$

$$h_a = \cdot 14 \quad h = 3 \text{ ft.}$$

$$Q = 4000 = 100 \left[3 \cdot 07 \left\{ (3 \cdot 14)^{\frac{3}{2}} - \cdot 14^{\frac{3}{2}} \right\} + 6 \cdot 4 d (3 \cdot 14)^{\frac{1}{2}} \right]$$

$$d = 2 \text{ feet}$$

height of anicut required $= 10 - 2 = 8$ feet.

Example 8.—The maximum flood discharge of a river is estimated at 5,000,000 cubic yards per hour, the mean velocity being 500 feet per minute. An anicut is to be built across the river, 450 feet long, with its crest $4\frac{1}{2}$ feet above bed. What must be the height of wing walls and head sluice, so that maximum flood may not rise within 3 feet of their tops?

Discharge per second $Q = \dfrac{5{,}000{,}000 \times 27}{60 \times 60} = 37{,}500$ Cusecs.

The velocity $= \dfrac{500}{60} = 8 \cdot 33$ feet per second.

Water section $A = \dfrac{Q}{V} = \dfrac{37{,}500}{8 \cdot 33} = 4{,}500$ Sq. feet.

Mean depth $= \dfrac{4{,}500}{450} = 10$ feet.

d = depth of water on crest of weir $= 10 - 4 \cdot 5 = 5 \cdot 5$ ft.

The velocity of approach is not known. Assume it to be equal to the mean velocity of the river = 8·33 ft. per second. The velocity head

$$h_a = \frac{V^2}{2g} = \frac{(8\cdot33)^2}{64} = 1\cdot08 : \quad . \quad \therefore (h_a)^{\frac{3}{2}} = 1\cdot12$$

see formula 30 of article 51.

$$Q = 37{,}500 \text{ cusecs} = L \left[3\cdot07 \left\{ (h + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}} \right\} + 6\cdot4d\,(h + h_a)^{\frac{1}{2}} \right]$$

Now assume

$$(h + h_a)^{\frac{1}{2}} = x$$

$$\therefore 37{,}500 = 450 \left[3\cdot07\,(x^3 - 1\cdot12) + 6\cdot4 \times 5\cdot5\,x \right]$$

$h_a = 1\cdot08$ assumed

$L = 450$ feet

$h_a^{\frac{3}{2}} = 1\cdot12$

$\therefore x = 1\cdot9$;

$\therefore x^2 = 1\cdot9^2 = 3\cdot61 = h + 1\cdot08$

$\therefore h = 2.53$

We have assumed $x = (h + h_a)^{\frac{1}{2}}$ or $x^2 = h + h_a$

Top of wing walls above the crest of the weir = 5·5 + 2·5 + 3 = 11 feet

Top of wing walls above the bed of the river = 11 + 4·5 = 15·5 feet.

Example 9.—An anicut 1,500 feet long is to be built across a river. A channel takes off from the river just above the site of anicut, and a head sluice controls the supply to the channel, in which the full supply depth above the floor of the sluice is 7·0 ft. and the area of sluice opening is such that a head of 6 inches is requisite to pass the given supply. The normal water section of the river is 7,500 sq. feet and the estimated normal discharge 30,000 C. ft. per second. What should be the height of crest of anicut above level of sluice floor, the latter being at the same level as bed of the river ?

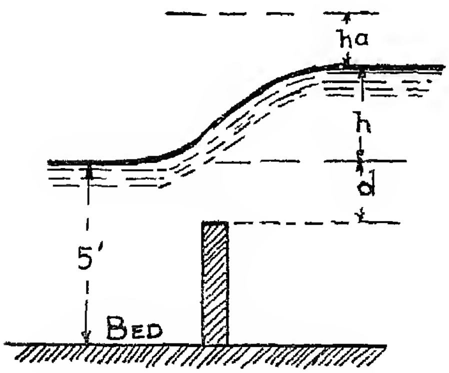
The mean depth of water in the river $= \frac{A}{L} = \frac{7{,}500}{1{,}500} = 5$ ft.

The mean velocity in the river $= \frac{30{,}000}{7{,}500} = 4$ ft. per Sec.

The water surface has to be raised through (7·0 — 5·0) + 0·5 = 2·5 ft.

$$Q = 5{,}000 = L \left[3\cdot07 \left\{ (h + h_a)^{\frac{3}{2}} - h_a^{\frac{3}{2}} \right\} + 6\cdot4d\,(h + h_a)^{\frac{1}{2}} \right]$$

Fig. 40 B



See formula 30 of article 51

h_a = Velocity of approach

$$= \frac{V^2}{2g} = \frac{4^2}{64} = \cdot25$$

$L = 1{,}500$ feet

$h = 2\cdot5$ feet

$$5{,}000 = 1{,}500 \left[3\cdot07 \left\{ (2\cdot75)^{\frac{3}{2}} - (\cdot25)^{\frac{3}{2}} \right\} + 6\cdot4d\,(2\cdot75)^{\frac{1}{2}} \right]$$

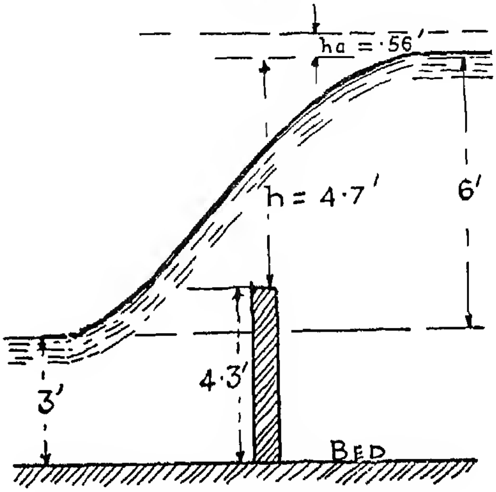
$\therefore$ d = ·6 ft.

$\therefore$ Height of crest above sluice floor = 5·0 — 0·6 = 4·4 ft.

If d had come out negative, it would indicate that the anicut crest must be above tail water.

Equation 29 of article 50 for a clear overfall should in this case be solved for h. Then 7·5 — h will give height of anicut above sluice floor.

Fig. 40 C



Example 10.—A jungle stream has a depth of 3 ft. and a mean velocity of 12 feet per second. What must be the height of anicut to raise the water level 6 ft. supposing that the bed silts up higher up the anicut, so as to give a total depth of water of 6 feet only.

It is evident that the crest level will be above tail water *i.e.* the anicut has a clear over-fall.

$$Q = \tfrac{2}{3}\, C\, L\, \sqrt{2g}\left\{ (h + ha)^{\frac{3}{2}} - ha^{\frac{3}{2}} \right\}$$

also $Q = 3L \times 12$

$$= \tfrac{2}{3} \times \cdot 57\, L\, 8 \left\{ (h + \cdot 56)^{\frac{3}{2}} - \cdot 56^{\frac{3}{2}} \right\}$$

h = 4·71 ft.

Height of anicut

= 6—4·7 + 3 = 4·3 above bed.

L = Width of stream.

$Q = A \times V$

$= 3L \times 12$

$$Va = \frac{3L \times 12}{6\,L} = 6 \text{ ft. per sec.}$$

$$ha = \frac{Va^2}{2g} = \cdot 56 \text{ feet.}$$

C = ·57.

Example 11.—A weir in a thin wall is 4 feet high and h is one foot. The bed of stream becomes filled up so that the depth of water over the weir becomes 2·5 feet instead of 5 feet, but Q the discharge is unaltered. How is h affected. First case: Complete contraction all round the weir

C = 0·62 L = Length of weir.

Wetted perimeter = 1 + 4 + L + 1 + 4 = L + 10 = m. See equation 9a : Article 20.

$$\text{Discharge } Q = \tfrac{2}{3}\, CL\, \sqrt{2g} \times h^{\frac{3}{2}}$$

Second case when the bed of channel is silted up and contraction is partly suppressed.

The wetted perimeter is $= L + 2 \cdot 5 + 2 \cdot 5 = L + 5$ feet. The length of the perimeter where contraction is suppressed $= (L + 10) - 5 = L + 5$ ft. $= n$ $\therefore \frac{n}{m} = 0 \cdot 5$

$C^1 = 0 \cdot 62 (1 + \cdot 14 \times \cdot 5) = 0 \cdot 6634$

$Q = \frac{2}{3} \times 0 \cdot 6634\, L \sqrt{2g}\, h_1^{\frac{3}{2}}$: h_1 = head over weir in 2nd case.

It is given that Q is constant in both cases.

$$\therefore \frac{2}{3} \times 0 \cdot 62\, L \sqrt{2g} \times 1^{\frac{3}{2}} = \frac{2}{3} \times 0 \cdot 6634\, L \sqrt{2g}\, h_1^{\frac{3}{2}}$$

$$\therefore \cdot 62 \times 1^{\frac{3}{2}} = \cdot 6634 \times h_1^{\frac{3}{2}}$$

$$\therefore h_1^{\frac{3}{2}} = \frac{0 \cdot 62}{0 \cdot 6634} = 0 \cdot 93 \text{ feet}$$

52. *Waterfall.*—The measurement of the discharge of a fall such as occurs at the end of a gutter or launder and in a more irregular fashion in a natural waterfall, is of importance. The discharge can be regarded as occurring over a very wide crested weir.

$Q = 4 \cdot 74\, L\, h^{\frac{3}{2}}$, where h is measured close to the fall on top of the crest.

53. Miscellaneous Remarks about weirs.

(*i*) Certain experiments on Bari Doab canal in Punjab, on a weir with flat top and sloping apron (1—10) 80 ft. long, when compared with rod float observations, gave the following results:—

$Q = 3 \cdot 49\, L\, h^{1.5}$

h = Observed head varying between $2 \cdot 5$ & $4 \cdot 2$.

Similar experiments on other weirs of same shape gave the following results:—

L	C	
29 ft.	3·52	V = 2 to 3 ft. per second.
37 ,,	3·54	h ranging from 3 to 3·5 ft.
46 ,,	3·58	V = 2 to 3 feet.
50 ,,	3·59	

(*ii*) Mr. Chatterton ("Hydraulic experiments on Kistna Delta in Madras Presidency"), worked on flat topped weir 2 feet wide, with P = 2 to 3 feet and L = 18 to 30 feet and found by current meter observations that

$Q = CLh^{1.5}$ P = Height of top of weir above bed of river.

C = $2 \cdot 99$ to $3 \cdot 21$

h = observed head = 3 to $4 \cdot 5$ ft.

Chatterton also experimented on Kistna anicut, flat top 6 ft. wide: sloping apron 1—10 and sloping approach 1—5 Length 3,500 feet. He found as follows:—

(*a*) with tail water below the crest.

$Q = 3 \cdot 13\, Lh \sqrt{(h + 0 \cdot 035\, V^2)}$

h was $4 \cdot 65$ feet during the above experiment.

(*b*) with tail water above the crest

$$Q = 3{\cdot}09\, L \left\{ (h + hv)^{1.5} - hv^{1.5} \right\} + C_2\, L\, d_1 (h + hv)^{\frac{1}{2}}$$

h = difference of level between head and tail waters.
d = depth of tail water over crest.
V = velocity of approach
hv = velocity head $= \dfrac{V^2}{2g}$
C_2 = coefficient $= 4{\cdot}90 + {\cdot}32$ d.

During the actual experiment, h varied from 2·72 to 3·67 and d, from 8·70 to 4·48, V from 5·21 to 3·73, C_2 from 7·65 to 6·05.

Lewis at Rasul, head works of Jhelum canal in Punjab, on a flat topped drowned weir 3 feet wide with P = 3 ft., h = 0·7 foot, d = 5·5 ft., found a discharge equivalent to that obtained by putting $C_2 = 6{\cdot}57$ in the above formula. The expression $C_2 = 4{\cdot}90 + {\cdot}32d$, gives 6·66. It is therefore probable that the above formula in the case of flat topped weir is not far removed from the correct value.

(*iii*) For flat topped weir with an apron sloping downstream, the formula

$$Q = 3{\cdot}50\ L\ h^{\frac{3}{2}}$$

appears to be well established.

" Horton's weir experiments, coefficients and formulae " is one of the best books on the subject.

54. *Sluices.*—A sluice is an aperture provided with a gate or shutter. Generally there are adjuncts which complicate the case and render the coefficient of discharge uncertain; when the gate is fully open, the case may approximate to that of an orifice in a thin wall. When it is nearly closed, the case may resemble that of a Prismatic tube ; where accuracy is desired, the coefficient must be determined experimentally. It may have any value from 0·50 to 0·80 or even outside these limits.

Sluices are constructed in many forms. A good idea of head-sluices, regulating the admission of water to channels can be obtained by visits to Head-works of (1) Jhelum canal at Rasul, Bari-Doab canal, Chenab canal in Punjab, Barrage canals at Sukkur in Sindh, Ganges canal at Haridwar and various other places.

Sluice openings or vents as they are called are separated from one another by piers which are generally provided with cut-waters. The sluice floor generally is at the same level as the bottom of the canal or the river. The bottom and side contractions are greatly suppressed, the coefficient may be taken as 0·80.

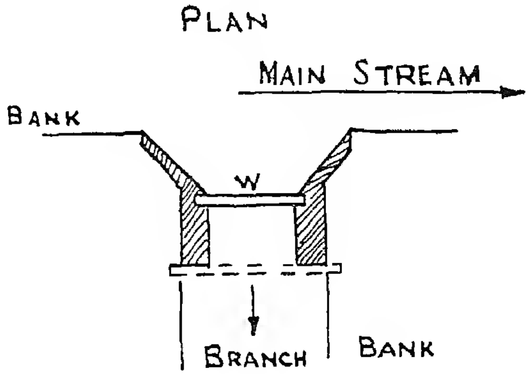
The water ways of river bridges and those in railway embankments across low ground, liable to floods, may be regarded as sluices, with a coefficient 0·8 or higher.

In all these cases the discharge takes place into water and the head is the difference of level between the water surfaces above and below the sluices.

Escape sluices.—These are rectangular openings in overflow weirs of large storage tanks or artificial lakes to allow the surplus flood water to escape, generally freely in the air. They are closed by vertically sliding shutters. At Khadakwasla dam near Poona, the shutters are provided with small wheels at sides, rolling on vertical rails, and closed or opened automatically by a large float in a well nearby. In some cases, as at Sukkur in Sindh, stoney gates operated by electric motors are used. The coefficient of discharge in such cases is 0·62.

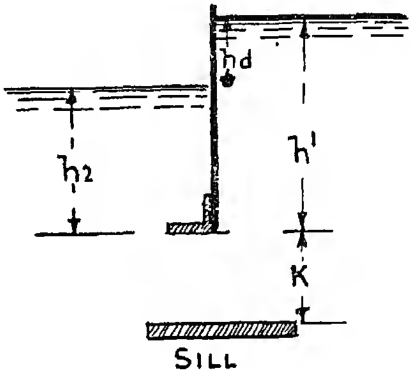
Some times, Tanks or artificial lakes are constructed by building a bund across valleys and water for irrigation purposes is let out through masonry rectangular arched culverts built in the bottom of the bund. Such a culvert has a shutter at its inner and upper end to control the discharge of water. The reader is advised to visit and see the outlets for canals, distributaries and other channels in various places to get a good idea of sluices.

Fig. 41



The following observations are taken from "control of water by Parker, page 164." The sluices and gates may be considered as orifices, usually rectangular in shape, with completely suppressed contraction along the lower portion of the perimeter more or less suppressed contraction at the sides and complete contraction at the upper portion. The problem is that of a submerged rectangular orifice.

When the gate is raised somewhat above the floor and the width of the opening is not too great, marked contraction at the bottom of the orifice is produced. The value of C, coefficient of discharge, is then 0·63. As the gate rises, the bottom contraction had less effect, the coefficient C increases upto 0·85, hd is maximum when k is minimum and *vice versa*.



Sir John Benton gives $C = 0{\cdot}7201 + {\cdot}0074\,w$ in the formula

$$Q = CA\sqrt{2ghd} \qquad \ldots\ldots 31.$$

W = width of gate ; and hd exceeds 0·5 ft.

velocity of approach neglected.

Discharge is submerged.

A = wk, the area of the opening when the gate is raised.

C = 0·63 if the discharge is not submerged.

Example 12.—The following are the levels in connection with the head-works of a canal, the full supply of which is 600 c. ft. per second.

Floor of sluice	43·26
Full supply level of canal (F. S. L.)	51·26
Crest of raised weir	51·26

Determine the number of vents or sluice openings, each 4 feet wide and 6 feet high which will be required for the head sluice.

Let n be the number:

A, the area of waterway $= n \times 6 \times 4 = 24\,n$.

The head $h = 51 \cdot 56 - 51 \cdot 26 = 0 \cdot 3$ feet.

$\therefore$ Discharge $Q = 600 = CA\sqrt{2gh} = 0 \cdot 8 \times 24n\ 8\sqrt{0 \cdot 3}$;

$C = \cdot 80$

whence $n = 7$

Example 13.—If in the preceding example, the depth of water flowing over the raised weir, is 10 feet above the crest, find the height to which the shutters should be raised off the sill, to pass off 600 c. ft. per second.

Let X be the height.

$A = 7 \times 4 \times X$; $h = (51 \cdot 56 + 10) - 51 \cdot 26 = 10 \cdot 30$ ft.

$Q = 600 = CA\sqrt{2gh} = 0 \cdot 8 \times 28 \times X \times 8\sqrt{10 \cdot 3}$

$\therefore X = 1 \cdot 04$ ft.

Example 14.—A surplus sluice in a tank weir consists of 8 vents each 4 feet wide. When there is 9 feet of water on the sluice sill, find the discharge per second if the shutters are lifted 5 feet and the discharge taking place in the air.

See article 31 .... equation 15.

$Q = \frac{2}{3} CL\sqrt{2g}\,(h_1^{\frac{3}{2}} - h_2^{\frac{3}{2}})$ $\qquad L = 4$

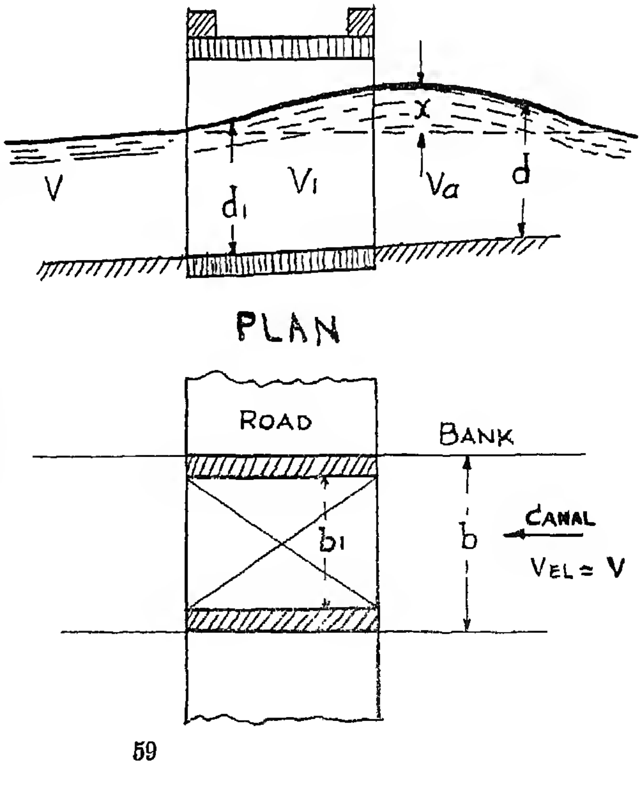
$= \frac{2}{3} \times \cdot 62 \times 4 \times 8\,(27 - 8)$ $\qquad h_1 = 9$

$\qquad h_2 = 4$

$= 251 \cdot 2$ cusecs through one vent.

$\therefore 8 \times 251 \cdot 2 = 2{,}010$ cusecs through all vents.

Fig. 42



55. *Afflux.*—A visit to a road Bridge over a canal or a raised weir across a river will show that water heads up on the upstream side of the Bridge to produce an increase of head X called Afflux as shown in the sketch, to create an increased velocity V_1 under the bridge to pass off the total discharge through the contracted water way of the bridge. The amount of afflux in most cases is small and can be approximately obtained from the formula

$$X = \frac{V^2}{2g}\left(\frac{1 \cdot 1\, b^2}{b_1^2} - 1\right) \quad \ldots\ldots\ldots\ldots\ldots \quad 32.$$

where V is the velocity of the canal, b the natural breadth and b_1 the contracted breadth under the bridge. The velocity of approach V_a, being taken equal to V (Love's Hydraulics).

The discharge is constant, under the bridge and outside the bridge on both sides.

$Q = CV_1 \times b_1 d$ under the bridge : $C = 0 \cdot 95$

also $= vbd.$
also $= V_a \times b\,(d + x)$ } ha = velocity head due to velocity of approach V_a.

V_1 is due to head $x + h_a = \sqrt{2g\,(x + h_a)}$

Moles Worth's pocket-book page 334 gives

$$x = \left(\frac{V^2}{58 \cdot 6} + 0 \cdot 05\right)\left\{\left(\frac{A}{a}\right)^2 - 1\right\} \quad \ldots\ldots\ldots\ldots\ldots \quad 33.$$

V = Natural velocity of canal.

A = Natural water section.

a = Water section under the bridge.

x = Rise of water due to obstruction

also velocity caused by any obstruction $= 1 \cdot 1 \dfrac{A}{a} V.$

In case of a weir across a river, " Afflux " means the difference of level between the high flood levels upstream and downstream of the river. The crest of the weir must be at such a level and of such a length so that when flood discharge passes over the weir, the Afflux is not so great as to overtop the canal regulator or the training banks upstream of weir.

The case is analogous to a submerged notch in article 36, h and h_1 in fig. 32 correspond to X and $(d + X)$ in fig. 42.

Example 15.—A bridge of seven arches of 20 feet span is constructed across a river which has in flood a mean width of 200 feet, mean depth of 6 feet and mean velocity of 5 feet per second. Find the afflux :—

See equation 32

$$X = \frac{V^2}{2g}\left(\frac{1 \cdot 1 b^2}{b_1^2} - 1\right) = \frac{25}{64}\left\{1 \cdot 1 \times \left(\frac{200}{7 \times 20}\right)^2 - 1\right\} = 0 \cdot 54 \text{ feet.}$$

56. *Bridge openings.*—A visit to a road bridge or a railway bridge across a river or a canal will make the following remarks clear to the reader.

The abutments and the piers of the bridge form an obstruction to the flow of water in the river. The water heads up on the upstream side and forces its way through the arched openings between the piers and abutments. These openings act as contracted channels, see fig. 42.

The coefficient C may have any value from 0·5 to 0·95 being smallest when the angles of the openings are sharp and square (Specially when the water section of the contracted channel is decreased both vertically and laterally), greater if the angles are champhered and curved. The coefficient is greater for large than for small openings.

In figure 42, it will be seen that the waterway under the bridge is smaller than the water section of the canal above it.

This obstruction is measured by difference in the upstream and downstream levels. This difference is very often inconsiderable.

'X' the heading up is most likely to be considerable with high discharges. When the stream leaves the contracted channel, eddies and scour occur.

The discharge coefficient C, mentioned above varies from 0·5 to 0·95; roughly for narrow openings, for square piers 0·6; obtuse angled 0·7; curved and acute 0·8. For wide openings add 0·1 (Bellasi's Hydraulics page 106).

In the fig 42, discharge under the bridge,

$$Q = CA_1 \sqrt{2gx} \quad \ldots\ldots 34.$$

When V_a, velocity of approach is neglected.

$$\left\{ \begin{array}{l} C = 0{\cdot}9 \\ A_1 = \text{Area of water section under the bridge.} \\ X = \text{Afflux in feet.} \end{array} \right.$$

$$Q = CA_1 \sqrt{2g\,(x + ha)} \quad \ldots\ldots 35.$$

ha = head due to velocity of approach.

In some cases, the width of the contracted channel has been constructed as equal to the bed width of the canal plus the full supply depth. The downstream part has been pitched to avoid Scouring, specially when the velocity is 5 feet per second or more.

Some authorities consider a bridge opening equivalent to a submerged weir and they deal separately with the discharges through sluice and notch portions of the area.

Example 16.—The country on each side of a railway embankment, which crosses low drainage line, is flooded. A water way of 25 feet discharges with depths of 6 feet on upper side and 4 feet on lower side. Estimate the discharge.

A_1 = Area of water way = $25 \times 4 = 100$ sq. feet.

Head $X = 6 - 4 = 2$ feet

Discharge $Q = CA_1 \sqrt{2g\ X}$ see formula 34.

$= 0{\cdot}9 \times 100 \times 8 \times \sqrt{2} = 1{,}018$ c. ft. per sec.

Example 17.—Find the afflux produced in a river 200 ft. broad, having velocity of 5 miles per hour, when crossed by a bridge, having four spans with piers 6 feet wide.

Four spans mean 3 piers; $3 \times 6 = 18$ ft. total width of piers; $200 - 18 = 182$ ft. clear water way $= b_1$

Velocity $V = \dfrac{5 \times 5280}{60 \times 60} = 7\ 30$ ft. per second

width of River = 200 ft. = b

See formula No. 32 : Afflux $X = \dfrac{V^2}{2g}\left(\dfrac{1{\cdot}1\,b^2}{b_1^2} - 1\right) = \dfrac{7{\cdot}3^2}{64}\left(\dfrac{1{\cdot}1 \times 200^2}{182^2} - 1\right) = 0{\cdot}28$ feet or 3 3 inches.

Example 18.—A river, whose mean width is 50 feet, depth 10 feet and mean velocity 3 ft. per second, has a bridge built across it. The piers and abutments are square and total width of water way in the bridge is 30 feet. Find the heading-up caused by the bridge:—

Discharge $Q = 50 \times 10 \times 3 = 1{,}500$ cusecs.

Water way in the bridge is $30 \times 10 = 300$ sq. feet.

$$\left.\begin{array}{ll} Q = 1500 = C\,A_1\,V_1 & \\ \quad = 0{\cdot}60 \times 300 \times V_1 & \\ \therefore\ 8{\cdot}33 = V_1 & \\ \text{Since } V1 = \sqrt{2gx} & \\ X = 1{\cdot}1 \text{ feet} & \end{array}\right\} \begin{array}{l} A_1 = \text{water way in the bridge} = 300 \text{ sq. ft.} \\ V_1 = \text{Velocity under the bridge.} \\ C = 0{\cdot}60 \text{ only because the piers are square:} \end{array}$$

Velocity of approach being neglected. If velocity of approach (given as 3 ft. per second) be taken into account, then by formula 35.

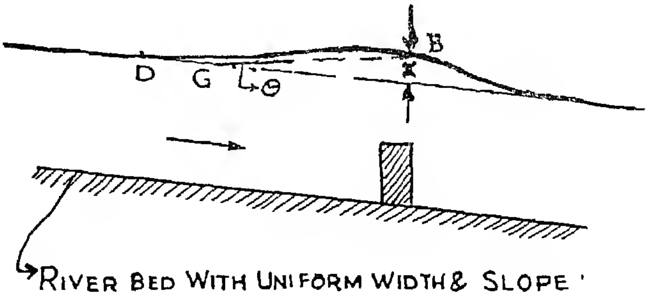
$$V_1 = \sqrt{2g\ (x + ha)}.$$

$$= \sqrt{2g\ (x + \tfrac{1}{7})}. \qquad ha = \text{head due to velocity of approach} = \tfrac{1}{7}.$$

$$\therefore\ 8{\cdot}33 = 8\sqrt{x + \tfrac{1}{7}}$$

$$X = 0{\cdot}938 \text{ feet.}$$

Fig. 42 A



57. *Backwater.*—Fig. 42A shows a weir placed across a river of uniform width and uniform bed slope: X is the afflux in feet. Water heads up in front of the weir, overtops it and falls again in the river, the new water-surface touches the normal water-surface of the river at point D. From observations it has been found that the length of the back-water B D is given by the formula

$$B\,D = 1{\cdot}5 \text{ to } 1{\cdot}9x \operatorname{cosec} \theta \qquad \ldots\ldots 36.$$

θ being surface fall.

It may be noted that back-water conditions produce deposit of silt in front of the weir, if the water in the river is charged with silt.

At Sukkur, the bed of the river rises 6″ annually due to silt deposit, occasioned by the Barrage, largest in the world. The banks of the river have to be raised often at a great expense to prevent floods overtopping the banks.

Example 19.—A stream of uniform breadth and slope has a normal depth of water $2\frac{1}{2}$ feet and a fall of 2 feet per mile. Find the length of back-water caused by a weir which raises the surface by $3\frac{1}{2}$ feet.

Length required $= 1{\cdot}5\, x \operatorname{cosec} \theta$ $\qquad x = 3\frac{1}{2}$ ft.

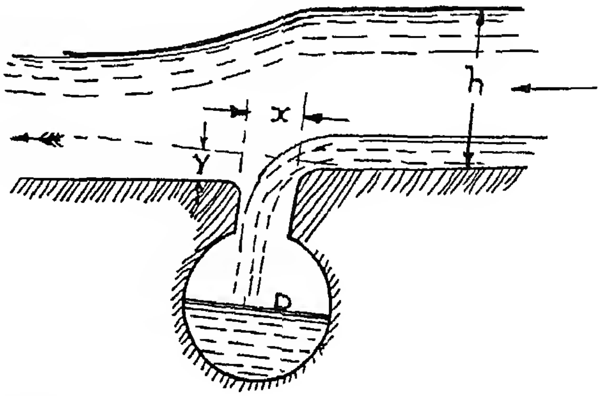
$$\operatorname{Cosec} \theta = \frac{5280}{2}$$

$$= \tfrac{3}{2} \times \tfrac{7}{2} \times \frac{5280}{2} = 13{,}860 \text{ feet}$$

$$= 2{\cdot}62 \text{ miles.}$$

58. *Separating weir.*—Fig. 43 shows a separating weir as used for water supplies of towns or for drains; when the discharge in the drain is normal, water falls in the collecting drain D.

Fig. 43



During times of rain, discharge increases to depth h, the velocity increases and water sheet jumps across the opening and passes into the waste channel :—

The mean velocity of the water sheet of depth h, when it leaves the small drain, is $\frac{2}{3}\sqrt{2gh}$: Let t be the time in seconds for the water sheet to jump across the distance X, then

$X = \frac{2}{3}\sqrt{2gh} \times t$: And if Y be the vertical fall

$$Y = \frac{gt^2}{2}$$

$$\therefore Y = \frac{9\,x^2}{16\,h} \qquad \ldots\ldots 37.$$

This gives Y for any assigned values of x and h (Love's Hydraulics).

59. *Modules.*—In some irrigation districts, the farmers are charged for water according to the area cultivated. In others, water is sold by volume and module is an apparatus employed to measure the quantity issued. In this case the cultivator has personal interest and does not waste water.

When a distributary takes off from a canal, a wooden sluice moving vertically in grooves in masonry pillars regulates the supply.

To secure a constant supply with a variable head the sluice is raised or lowered as required.

Sometimes a notch with a gauge is built in masonry chamber, a little away from the sluice to measure the flow.

In some places, a masonry chamber is built and a conical plug put in a hole and operated by a float which rises or falls according to the water level in the canal. When the water level in the canal rises, the float goes up, the plug goes down and the annular space between the plug and hole is shortened and the disoharge is reduced *vice versa.*

In Punjab in India, Mr. Kennedy, the Chief Engineer (1900) invented a module and used it freely on the canals. Later on Mr. Crump, an officer in the same department invented a module called 'Adjustable Proportionate Module' and used it in the Irrigation Department. Details are not given here as the matter will cover more than one dozen pages and several drawings.

Paper No. 237 "Improved Adjustable, Proportional modules and open flumes" by Pandit K. R. Sharma, P.S.E., A.M.I.E. (India), an officer in Punjab Irrigation Department, was read before the Punjab Engineering Congress in 1940. Full particulars about this module and others by Kennedy, Gibbs and some more authors can be had from the Secretariat, Punjab Irrigation Department, Pakistan or Hindustan.

Example 20.—A submerged weir, 10 feet long, has a depth of water on the upstream side, of 17 inches, on the downstream side of 9 inches. The velocity of approach = 1.98 feet per second. Assuming the head equivalent to this velocity of approach to be given by $h = 1.4 \frac{V^2}{2g}$, determine the discharge in cubic feet per minute.

Assume C = 0.592

Answer 3,090 cubic feet per minute.

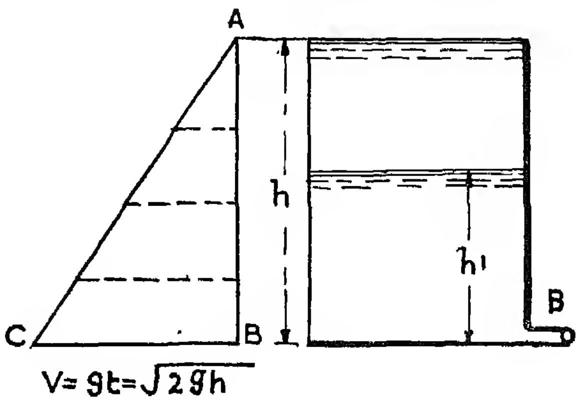
CHAPTER IX.

Discharge Under Variable Head, Orifices and Sluices.

60. Free discharge from Prismatic vessels.

The fig. No. 44 shows a prismatic vessel full of water. When an Orifice is made on one side near its bottom, the water flows out with a velocity, $V = \sqrt{2gh}$. If a drop of water falls vertically through height h in t seconds, the velocity at point B is gt.

Fig. 44.



The velocity is directly proportional to the head. As the water in the vessel sinks down, the velocity at point B decreases and reduces to zero when the head h is nill. The ordinates on line AB in the triangle ABC represent the velocities through the Orifices at different heights. When the orifice at B is open, the water level in the vessel does not sink down uniformly, *i.e*, through equal heights in equal times. The head goes on decreasing, so does the discharge, not by equal amounts in equal times.

The mean velocity through the time t seconds is $\frac{V}{2} = \frac{gt}{2} = \frac{(2gh)^{\frac{1}{2}}}{2}$

Hence the discharge through the orifice, when the head h decreases from h to zero, is half the discharge, which would take place in the same time under a constant head h.

The mean velocity is $\frac{1}{2}\sqrt{2gh} = \left(2g\frac{h}{4}\right)^{\frac{1}{2}}$.

It is evident that the mean head is $\frac{h}{4}$.

61. *Time of emptying or filling a vessel and filling one vessel from another.*—In the figure 44 article 60. Let A be the horizontal cross sectional area of the prismatic vessel; h is the maximum depth above the orifice at point B.

Let t be the time required to empty the vessel or of change of head from h to 0.

The mean velocity through the orifice is $\frac{1}{2}\sqrt{2gh}$. The mean discharge per second is $ca\frac{1}{2}\sqrt{2gh}$, a being the area of the orifice in square feet and c being equal to 0·60. The whole discharge from the orifice in 't' seconds $= ca\frac{1}{2}\sqrt{2gh} \times t$ cusecs.

The contents of the vessel = whole discharge = Ah.

$$\therefore t = \frac{2\,Ah}{ca\sqrt{(2gh)}} \qquad \text{.......... 38.}$$

This means that time 't' is double that which would be required to discharge the same volume under a constant head 'h'.

If the head diminishes from h to h^1 in time t seconds, t can be found as under.

$$\text{Time from h to 0 is } \frac{2\,Ah}{ca\sqrt{(2gh)}} \qquad \text{.......... (m)}$$

$$\text{Time from } h^1 \text{ to 0 is } \frac{2\,Ah^1}{ca\sqrt{(2gh^1)}} \qquad \text{.......... (n)}$$

$\therefore$ (m) — (n) = time, t^1 seconds, during which the head h diminishes to $h^1 = \frac{2\,Ah}{ca\sqrt{(2gh)}} -$

$$\frac{2\,Ah^1}{ca\sqrt{(2gh^1)}} : t^1 = \frac{2\,A}{ca\sqrt{(2g)}}\left(\sqrt{h} - \sqrt{h^1}\right) \qquad \text{.......... 39.}$$

If the discharge takes place through a pipe of cross sectional area a and length L feet and

$$\text{diameter d feet, then } t^1 = \frac{8\,A\left(1 + \frac{4\,f\,L}{a}\right)^{\frac{1}{2}}\left(\sqrt{h} - \sqrt{h^1}\right)}{\pi\,d^2\sqrt{2g}} \qquad \text{.......... 39a.}$$

f being 0·01

If t^1 is known, h or h^1 can be found if the other is known. Thereafter volume $A\,(h - h^1)$ or discharge in t seconds can be known.

If ha be an average head between h & h^1, then $t^1 = \dfrac{A(h - h^1)}{ca\ (2gha)^{\frac{1}{2}}}$

$$\therefore \sqrt{ha} = \frac{h - h^1}{2(\sqrt{h} - \sqrt{h^1})} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad 40.$$

If Va be an average velocity between h and h^1 in time t^1, that is, velocity which would, if it were uniform during the time t^1, reduce the level in the tank to the same extent in the time. V is the velocity with head h. Va = KV, K being the factor which bears the value given below for different values of $\dfrac{h^1}{h}$

$\frac{h^1}{h}$	K	$\frac{h^1}{h}$	K	$\frac{h^1}{h}$	K
·0	·50	·4	·816	·8	·947
·1	·658	·5	·854	·9	·974
·2	·724	·6	·887	1·0	1·000
·3	·774	·7	·918		

The equation No. 40 is useful for canal locks. If in figure 44, h^1 is zero, that is, if the vessel is emptied down to the level of the orifice, then equation (40) becomes

$\sqrt{ha} = \dfrac{\sqrt{h}}{2}$ or $ha = \dfrac{h}{4}$ or mean head is $\frac{1}{4}$ of h.

See the last line of article 60.

The following are ratios of $\sqrt{ha}$ to $\sqrt{h}$ for certain cases (Bellasis):

for a prism or cylinder .... $\frac{1}{2}$

for a sphere .... $\frac{5}{8}$

for a hemisphere concave downwards .... $\frac{5}{7}$

for a hemisphere concave upwards .... $\frac{5}{12}$

for a cone with apex downwards .... $\frac{5}{6}$

for a cone with apex upwards .... $\frac{5}{16}$

for a wedge with point downwards .... $\frac{3}{4}$

for a wedge with point upwards .... $\frac{1}{3}$

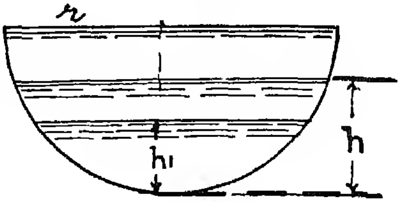
for a vessel whose vertical section is a parabola with vertex downwards when all vertical sections are the same (paraboloid of revolution) .... $\frac{3}{4}$

When the horizontal sections are rectangular (two opposite sides of the vessel are rectangular and two parabolas) $\frac{2}{3}$

In the last case the surface falls at a uniform rate, that is, in equal amounts in equal times.

In all cases, the times occupied in emptying the vessels are greater than with head ha, in the inverse ratios of the above fraction.

Fig. 44 A



Hemisphere vessel :—Radius $= r$: t (time) taken for water level to fall from h to h^1 when the orifice of area 'a' sq. feet is opened at bottom.

$$t = \frac{2\pi}{ca\,(2g)^{\frac{1}{2}}}\left\{\frac{2}{3}r\left(h^{\frac{3}{2}} - h^{1^{\frac{3}{2}}}\right) - \frac{1}{5}\left(h^{\frac{5}{2}} - h^{1^{\frac{5}{2}}}\right)\right\} \qquad \text{.......................40A.}$$

If the vessel is full at the commencement and is completely emptied, then $h = r$ and $h^1 = 0$. Equation 40 A becomes

$$t = \left(\frac{14\,\pi\,r^{\frac{5}{2}}}{15ca\,(2g)^{\frac{1}{2}}}\right) \qquad \text{.......................40B.}$$

If the vessel is filled through an orifice in its bottom, from a tank in which the water remains level with the top of the vessel, the ratio of $\sqrt{ha}$ to $\sqrt{h}$ is the same as for filling the vessel when inverted. Thus for a cylinder, prism, or sphere, the time for filling is the same as for emptying.

If two prismatic vessels communicate by an orifice or a pipe, and h be the difference in the water levels of the vessels and A_1 & A_2 their horizontal areas, the time which elapses before the two heads become equal is

$$t = \frac{2A_1\,A_2\,\sqrt{h}}{ca\left\{2g\,(A_1 + A_2)\right\}^{\frac{1}{2}}} \qquad \text{.......................41.}$$

This equation may be used for double locks.

$C =$ Coefficient $0 \cdot 6$ generally.

$a =$ Area of connecting pipe, length upto 25 times diameter.

If the tank be full of water and a hole is made in its bottom and also there is constant inflow in the tank at its top, find the time required to empty the tank.

a = area of orifice in the bottom of the tank in sq. feet.

q = discharge do. do. in cusecs : Q = constant inflow in cusecs.

A = area of tank in plan : t, time required to raise or lower liquid surface between h^1 and h^2, is given by the following formula.

$$t = \frac{2A}{K^2}\left[Q\log\left(\frac{Q - K\sqrt{h^2}}{Q - K\sqrt{h_1}}\right) + K\left(h_2^{\frac{1}{2}} - h_1^{\frac{1}{2}}\right)\right] \qquad \text{.......................41A.}$$

K being $= c\,a\,\sqrt{2g}$ and C being $= 0 \cdot 60$.

Example:—

A cylindrical vessel having a diameter of 5 747 inches has an orifice 0·2 inches in diameter, and the fluid surface is observed to sink from 16 inches to 12 inches in depth in 53 seconds (t_1).

Find the coefficient of discharge, taking $g = 32\cdot19$.

See formula No. 39.

$$C = \frac{2A}{t_1\, a\, (2g)^{\frac{1}{2}}} \left(h^{\frac{1}{2}} - h_1^{\frac{1}{2}} \right)$$

$$= \frac{2\left(\frac{5\ 747}{12}\right)^2 \times \frac{\pi}{4}}{53\ \frac{\pi}{4}\left(\frac{\cdot2}{12}\right)^2 (2g)^{\frac{1}{2}}} \left\{ \left(\frac{16}{12}\right)^{\frac{1}{2}} - \left(\frac{12}{12}\right)^{\frac{1}{2}} \right\} = 0\cdot60.$$

Example:—

A square prismatic basin, whose side is 3 feet, has an orifice 0·09 feet in diameter, 6 feet below the surface. Find the discharge in $4\frac{1}{2}$ minutes taking $C = 5/8$.

Equation 39 is

$$t^1 = \frac{2A}{ca\,(2g)^{\frac{1}{2}}} \left(h^{\frac{1}{2}} - h_1^{\frac{1}{2}} \right) : A = 3 \times 3 = 9 \text{ sq. feet}$$

$$C = \frac{5}{8} = \cdot625$$

$$h = 6 \text{ feet}$$

$$270 = \frac{2 \times 9}{\cdot625 \times \cdot0636 \times 8} \left(6^{\frac{1}{2}} - h_1^{\frac{1}{2}} \right) : t^1 = 270 \text{ seconds}$$

$$a = \cdot0636 \text{ area of orifice in sq. feet.}$$

$h_1^{\frac{1}{2}} = 1\cdot972$ feet

$h = 3\cdot89$: discharge required is $A\,(h - h_1)$

$= 9\,(6 - 3\cdot89) = 19$ cusecs.

62. *Canal Locks.*

Fig. 45

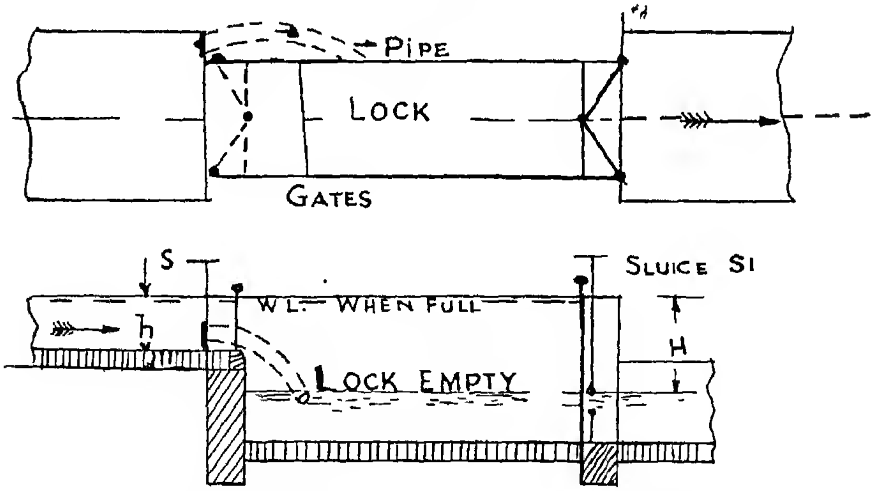


Fig. 45 is a sketch of a single lock. The writer saw a lock on the river Jhelum about two miles below the town of Srinagar in Kashmir State. There is a raised weir across the Jhelum river. A single lock is built at one end of the weir to admit of boats, carrying merchandise, to go up the river.

When the boat proceeds up the river, first it is admitted into the partly empty lock, the lower gates are closed, the sluice near the upper gate is opened and the lock is filled through the pipe.

When the water level is the same in the lock as in the upper part of the canal, the boat then proceeds up the canal. The process is reverse when the boat comes down the canal.

(*a*) Time required to empty the lock :

Let A be the area of water-surface in the lock. h = the depth from water surface of upper reach of canal to the centre of the sluice in the same reach. a and a_1 be areas of upper and lower sluice openings. H, called lift, is difference of water-surface in the upper reach and lower reach of the canal.

Close the upper gates and then open the lower sluice which is submerged, the head varying from H to O. By equation 38, time required to empty the lock.

$$t = \frac{2AH}{ca^1 \sqrt{2gh}} \quad \ldots\ldots 42.$$

(*b*) To fill the lock :

From the level of lower reach to the centre of the sluice in the upper reach, the depth is H — h. The orifice in the upper sluice discharges freely in the air. Equation 9 applies in this case.

$$t_1 = \frac{A(H-h)}{ca\,(2gh)^{\frac{1}{2}}} \; : \; C = 0{\cdot}60$$

From the centre of upper sluice to water level of upper reach, the depth is h : now the upper sluice functions as a submerged orifice.

The depth reduces from h to o. The time t_2 for this portion is given by the equation

$$38{\cdot}t_2 = \frac{2Ah}{ca\sqrt{2gh}}$$

Hence the total time $t = t_1 + t_2 = \dfrac{A(H+h)}{Ca\sqrt{2gh}}$ ......43.

Note :—If in the above case the head h reduces to h′ *i.e.* not to zero, which means that the water level rises only $h - h_1$ feet, the time t_3 can be found by the equation No. 39.

63. *Discharge from a prismatic vessel through a rectangular notch.*—Let t be the interval during which the head comes down from H to h, L be the length of the notch.

A = area of the vessel. C = ·62 for a thin plate and 0·577 for a broad crested weir

$$t = \frac{3A}{CL\sqrt{2g}}\left(\frac{1}{\sqrt{h}} - \frac{1}{\sqrt{H}}\right) \quad \ldots\ldots 44.$$

Example :—A Tank, the water spread of which is one-fourth of a square mile, is provided with a weir 60 ft. long which discharges with a maximum depth of water 3 ft. on its crest; supposing no water to enter the tank, find the time in which surface will be lowered by one foot.

This is the case of a big prismatic vessel discharging through a rectangular notch. See equation 44,

$A = \dfrac{5280 \times 5280}{4}$ L = Length of the weir 60 ft.

H = 3 feet; h = 2 ft. C = 0·58

see equation 44. $t = \dfrac{3A}{CL\sqrt{2g}}\left(\dfrac{1}{\sqrt{h}} - \dfrac{1}{\sqrt{H}}\right)$

$= \dfrac{3 \times 5280 \times 1320}{\cdot 58 \times 60 \times 8}\left(\dfrac{1}{\sqrt{2}} - \dfrac{1}{\sqrt{3}}\right)$ seconds.

54·4 minutes.

64. *Discharge from non-prismatic vessels.*—If the discharging vessel is non-prismatic, the ratio of the time of emptying under a variable head to the time under constant (average) head is no longer uniform.

Thus the ratio for wedge-shaped vessels is $1\frac{1}{3}$, that for a pyramidal vessel is $1\frac{1}{5}$, the base of the wedge or pyramid being the water-surface.

In the case of a paraboloid vessel, the ratio of the times is $1\frac{1}{3}$ or the same as that for a wedge-shaped vessel.

65. *Head uniformly varying.*—Let the head over an orifice during time t vary from H_1 to H_2 and let the discharge in this time be Q. The mean head H_a (or equivalent head) is that which would, if maintained constant during the time t, give the discharge Q. Let H vary uniformly that is by equal amounts in equal times, as for instance in the case of an orifice in the side of an open stream whose surface is rising or falling at a uniform rate.

$$\sqrt{H_a} = \frac{2}{3}\left(\frac{H_1^{\frac{3}{2}} - H^{\frac{3}{2}}}{H_1 - H_2}\right) \qquad \ldots\ldots 45.$$

If $H_2 = 0$, that is, if the head varies uniformly from H_1 to 0 or from 0 to H_1

$$\sqrt{H_a} = \frac{2}{3}\sqrt{H_1} \qquad \ldots\ldots 46.$$

or the equivalent head H_a is $\frac{4H_1}{9}$.

Examples:—A large tank has a circular sharp-edged orifice 1·44 sq. inch area at a depth of 9 ft. below constant water level. The jet issues horizontally, and in a horizontal distance of 7·8 ft. it falls 1·8 ft. The measured discharge is 0·15 cusecs. Calculate the coefficients of velocity, contraction, and discharge. (A.M.I. Civil E.)

Ans:—·97; ·643; ·624.

Compensation water is to be discharged by two circular orifices under a constant head of 2 ft. 6 ins., measured to the centre of the orifices. What diameter will be required to give 3,000,000 gallons a day? $C_c = \cdot 62$; $C_v = \cdot 97$. (A.M.I. Civil E.)

Ans:—8·18 ins.

Two vertical sided basins each having a surface area of 2,000 sq. ft., are connected by a sluice gate of area 2 sq. ft. The initial difference of level in the basins is 9 ft. How long will it take to reduce this to 4 ft.? The coefficient of discharge of the orifice is ·8. (A.M.I. Civil E.)

Ans:—2 mins. 30 secs.

Find the depth and top width of a triangular notch, capable of discharging a maximum quantity of 25 cusecs and such that the head shall be three inches when the discharge is 0·2 cusecs. For a right-angled notch, C = 2·54. (A.M.I. Civil E.)

Ans:—1·725 feet
8·7 feet

Fig. 45 A

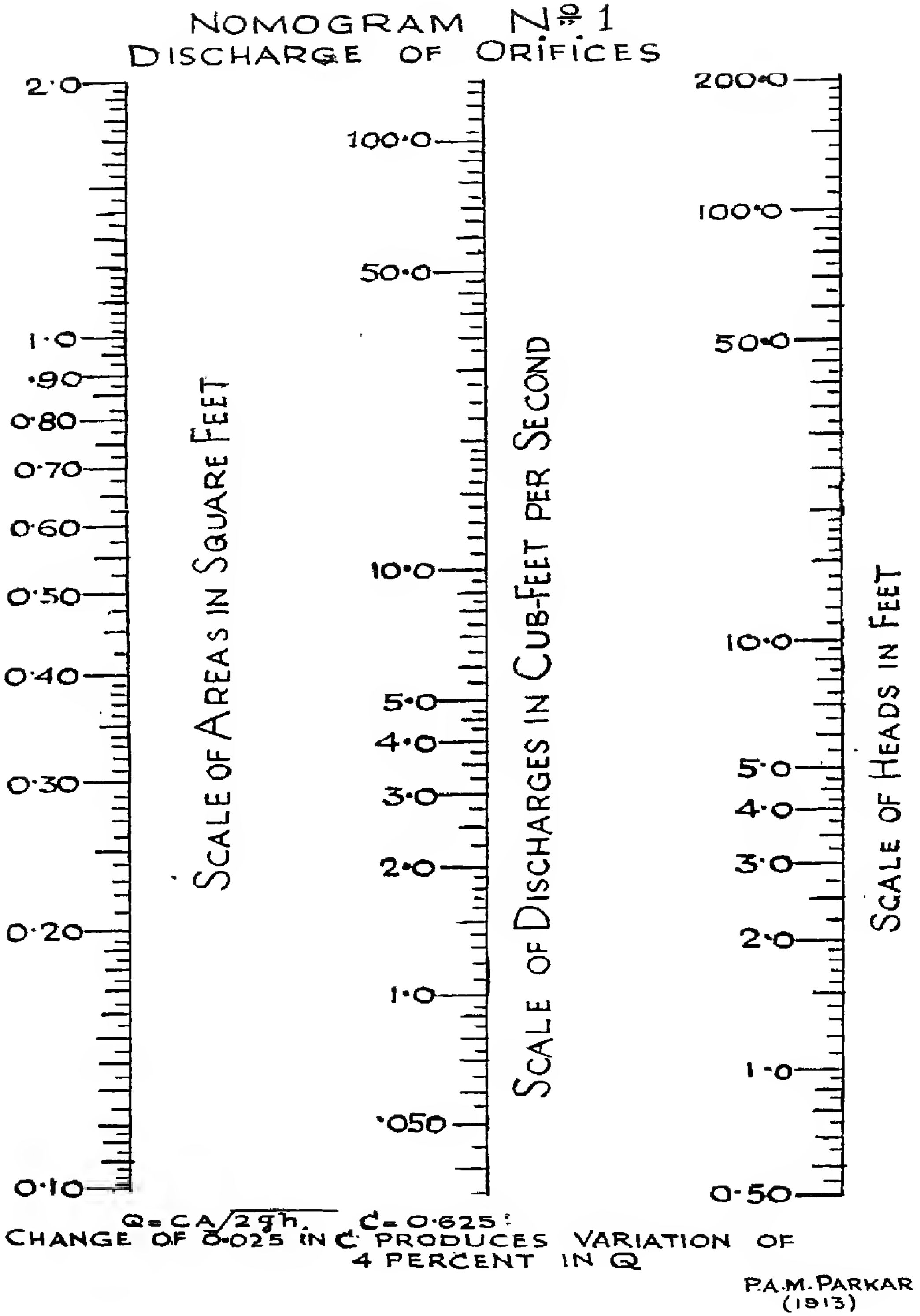
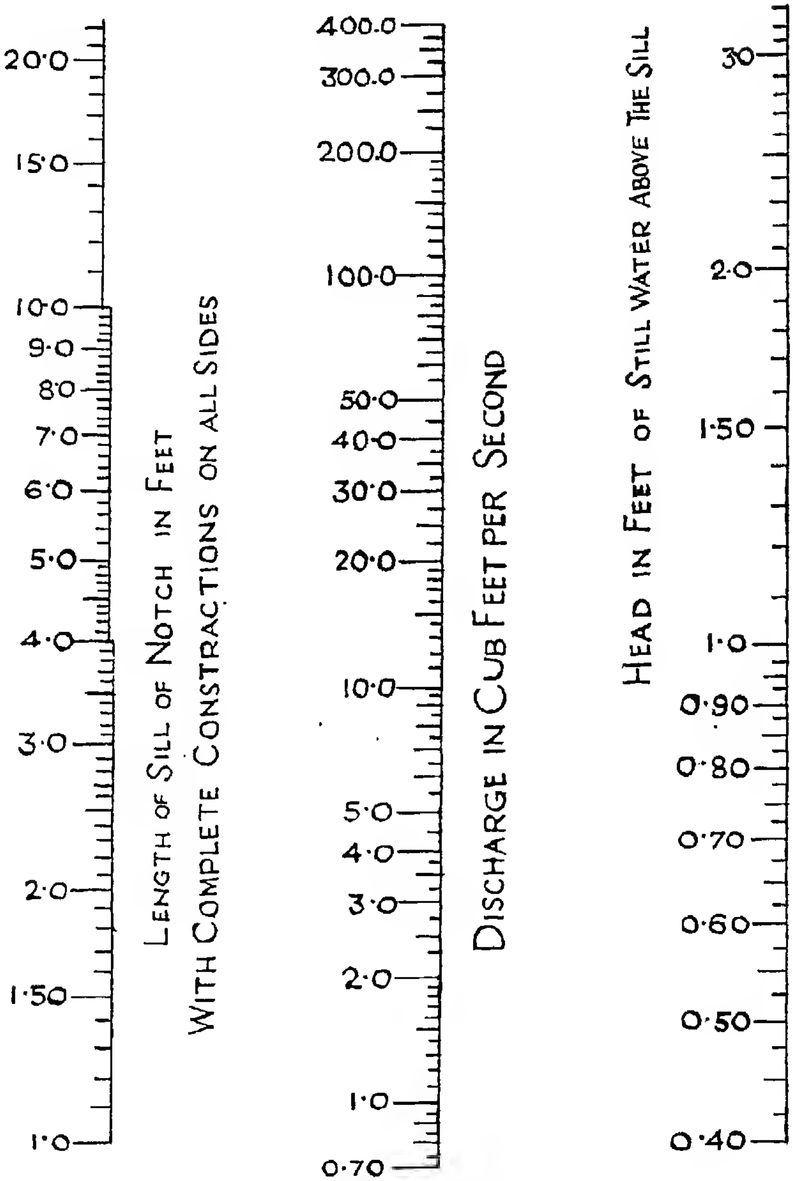

NOMOGRAM No 1
DISCHARGE OF ORIFICES
2·0
1·0
·90
0·80
0·70
0·60
0·50
0·40
0·30
0·20
0·10
SCALE OF AREAS IN SQUARE FEET
100·0
50·0
10·0
5·0
4·0
3·0
2·0
1·0
·050
SCALE OF DISCHARGES IN CUB-FEET PER SECOND
200·0
100·0
50·0
10·0
5·0
4·0
3·0
2·0
1·0
0·50
SCALE OF HEADS IN FEET
Q = CA√2gh. C = 0·625:
CHANGE OF 0·025 IN C PRODUCES VARIATION OF 4 PERCENT IN Q
P.A.M. PARKAR
(1913)

Fig. 45 B

NOMOGRAM No 2

DISCHARGE OF SHARP-EDGED WEIRES



$$Q = \frac{2}{3} C.L.\, h^{\frac{3}{2}} \sqrt{2g}$$

$$C = 0{\cdot}62$$

STILL WATER TANK

INCREASE h BY $\frac{v^2}{2g}$ IF VELOCITY OF APPROACH BE V

P.A.M. PARKER
(1913)

CHAPTER X.

Flow of Water in Pipes. Steady Motion.

66. *Fluid Friction.*—A pipe is a channel, elosed on all sides. Suppose a pipe is laid on ground with uniform slope. It is observed that when water enters the pipe, the velocity soon becomes uniform showing that the force due to gravity is exactly balanced by the resistance to motion offered by the inside boundaries of the pipe.

The frictional resistance is governed by the following laws, which differ very considerably from those governing the friction between solid bodies.

(*i*) The frictional resistance varies with the nature of solid surface but is independent of the pressure.

(*ii*) For large surfaces it is proportional to the area of the surfaees.

(*iii*) For velocities greater than one ineh a second, the amount of friction is proportional, very nearly, to the square of the velocity ; for velocities less than one inch a second. it is very nearly directly proportional to the velocity, also independent of nature of surface of contaet, because then the motion of liquid is stream line flow.

The velocity at which the flow changes from steady stream line flow to eddy flow is known as the critical velocity.

Let f be the frictional resistance for an area of one square foot at a velocity of one foot per second, then for an area a, and velocity V, the resistance $= \text{f}\ a\ \text{V}^2$, the value of f increasing with the roughness of the surface.

If $\mu = \dfrac{2\text{gf}}{\text{w}}$, we have total resistance

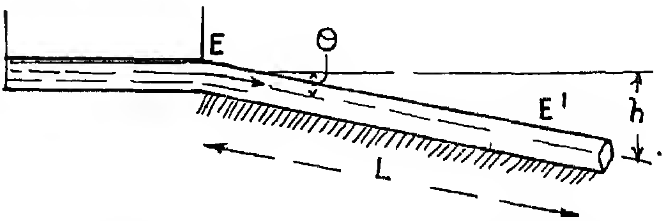
$$\text{R} = \mu \text{w}\, a \frac{\text{V}^2}{2\text{g}} \quad \text{...46A.}$$

μ is then called coefficient of friction, usually determined by experiment. Thus for well painted iron $\mu = \cdot 0049$. For incrusted C. I. pipes $\mu = \cdot 010$.

Some mathematicians have made very interesting experiments to find μ ; but these being elaborate, are not re-produced here.

67. *Velocity in Pipe.*—Fig. 46 shows a pipe resting on a uniformly sloping ground. It takes off from a reservoir with water surface at a constant level with the top of the pipe. Let d be the diameter of the pipe in feet. L = the length of the pipe in feet. h = the fall of ground surface or axis of the pipe in length L. A = area of the cross section of the pipe in sq. feet. B = The wetted perimeter of the pipe. The ends of the pipe are open and subject to atmospheric pressure only. There is only gravity flow in the pipe.

Fig. 46



When the column of water moves in the pipe through length L against the frictional resistance of the inside of the pipe, the accelerating force due to gravity is component of its weight, acting parallel to the axis of the pipe or $\text{WAL}\dfrac{\text{h}}{\text{L}}$ or WAh.

The surface offering resistance to flow of water is the length × wetted perimeter of the pipe = LB.

The total frictional resistance is equal to the total accelerating force : $fLBV^2 = WAL\frac{h}{L}$

or $V^2 = \frac{W}{f} \; \frac{A}{B} \; \frac{h}{L}$, f being 0·01 : approximately.

By equation 46 A, $\frac{W}{f} = \frac{2g}{\mu}$, μ being coefficient of friction : $\frac{A}{B} = \frac{\text{AREA}}{\text{Wetted Perimeter}}$ = Hydraulic radius called r : $\frac{h}{L}$ = sine θ = slope of the pipe, s ; let $\frac{W}{f} = C^2$, where C is a coefficient.

Then $V^2 = C^2 rs$ or $V = c\sqrt{rs}$ ....47.

In the above case if the inside circumference of the pipe were to be spread out in a straight line, length $2\pi r$, the depth of water will be $\frac{r}{2}$ feet, called hydraulic mean radius. Also the uniform slope of the ground or the pipe is called the Hydraulic gradient of the pipe and is equal to $\frac{h}{L}$ (sine of the slope), denoted by the letter s.

The velocity of water flowing along a pipe will vary at different points of the cross section, its magnitude depending on the radius. The velocity at any radius may be measured with a pitot tube.

It is found that velocity is maximum at centre and minimum at the circumference.

The maximum velocity* is about 1·2 times the mean velocity.

The mean velocity occurs at a point at a distance of 0·74 r from the centre, r being the radius of pipe. The velocity curve with diameter of pipe as a base is a semielipse according to some : others maintain that it is a parabola.

Fig. 47

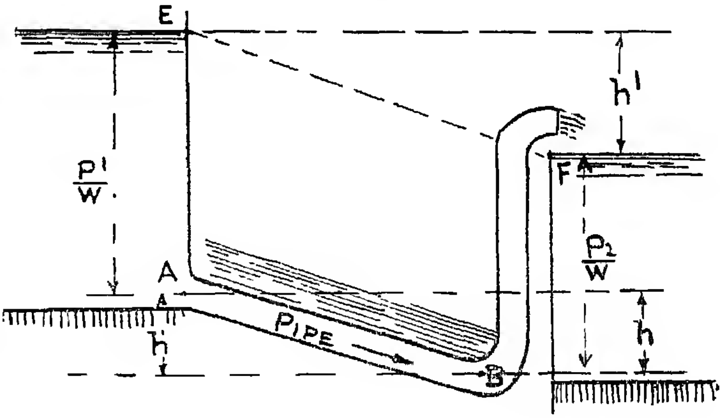


Fig. 47 shows a pipe taking off from a reservoir and discharging into another reservoir, the pressures at the two ends have to be taken into consideration as well. Let P^1 and P^2 be the pressures at two ends A & B. The hydraulic gradient line is E F. The resultant pressure on the mass of water AB, resolved parallel to the axis of the pipe, is

$$A(P^1 - P^2) \text{ or } WA\left(\frac{P^1}{W} - \frac{P^2}{W}\right) = WA\left(h^1 - h\right)$$

$$\text{since } \frac{P^1}{W} + h = \frac{P^2}{W} + h^1 \quad \ldots\ldots\ldots\ldots (a).$$

* For complete account of Reynold's experiments on velocities, see Philosophical transactions 1883.

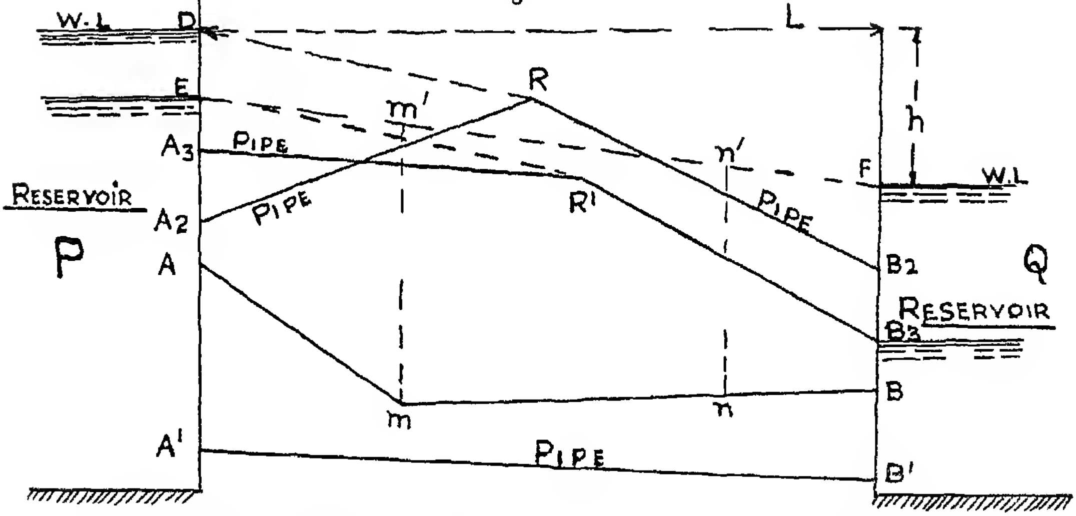
The component of the weight of water in pipe A B, parallel to the axis of the pipe, is W A h as before ....(*b*).

(a) + (b) = W A h¹ : this is equal to f L B V² or $V^2 = \frac{W}{f} \frac{A}{B} \frac{h^1}{L} = C^2 rs \therefore V = C \sqrt{r} \sqrt{s}$ same as formula (47).

68. *Entrance head.*—In the fig. 47, when the water enters the pipe, a portion of the initial pressure of the water is transformed into velocity and the pressure is reduced by $\frac{V^2}{2g}$ feet of water. The loss of head at entry into the pipe is independent of the length of the pipe and can be expressed by $\frac{V^2}{2g}$ (1 + a), where $a \frac{V^2}{2g}$ represents the resistance of the entrance of the pipe, considered as an Orifice, discharging water at a Velocity of V feet per second, *a* varying from about 0·06 for a pipe with a bell-mouthed entry to 0·50 for a pipe projecting into the reservoir.

In calculating the large syphon used on Punjab canals, it is usual to assume that loss at entry is $1 \cdot 5 \frac{V^2}{2g}$.

Fig. 48



69. *Virtual line of slope or Hydraulic gradient.*—Fig. 48 shows two reservoirs connected by a pipe line AB, of uniform diameter. When the water enters the pipe at point A, the pressure at A is reduced on account of loss due to "Entrance head", see article 68. Thus E is the effective point. Join E F. If pressure columns on pipe A B are put up at points M & N, the water would rise to M¹ and N¹; also the points M¹ and N¹ will lie on the line E F, discharge of water taking place through the pipe line A B.

The line E F is the hydraulic gradient. The difference of level between the water surfaces of the two reservoirs or the head lost due to friction in the pipe under discharge is h feet and the distance apart is L feet : $\frac{h}{L}$ is the hydraulic gradient slope, which has nothing to do with the slope of the pipe line A m n B or the ground surface.

In the equal parts of the length L, the falls in the line of gradient or losses of head by friction are equal.

Sometimes the length of the pipe line A B is so great that the loss of head at the entrance is neglected, and the length of the mouthpiece is also neglected. The actual position of the pipes is of no consequence: the virtual slopes of the pipes A B and A^1 B^1 are all equal, provided the pipes are of the same diameter, roughness and lengths.

The pressure on the virtual slope line E m^1 n^1 F is the same throughout and equal to atmospheric pressure only.

If the pipe line takes the course A_2 R B_2 and the point R is at a height not more than 34 feet above the virtual slope line E F, then the pressure inside the pipe at point R is less than atmospheric pressure, the pipes are in the condition of a partial syphon. At such a point in practice, the air may be disengaged from the water and the flow retarded, the line of hydraulic gradient being shifted to something like line DR, and the pipes R B_2 running only partly full.

If the height of the point R above the line Em^1 n^1 F be more than 34 feet, the pressure inside the pipe becomes negative and flow impossible. The above holds good specially if water in the reservoirs is subjected only to atmospheric pressure. But if the reservoir P has an airtight roof, and compressed air is introduced in it, the hydraulic gradient line will move up, over and above the point R; then the flow through the pipe at R will be full. Otherwise an air valve should be introduced at the point R to allow the air bubbles to escape. The best thing is to avoid the course A_2 R B_2.

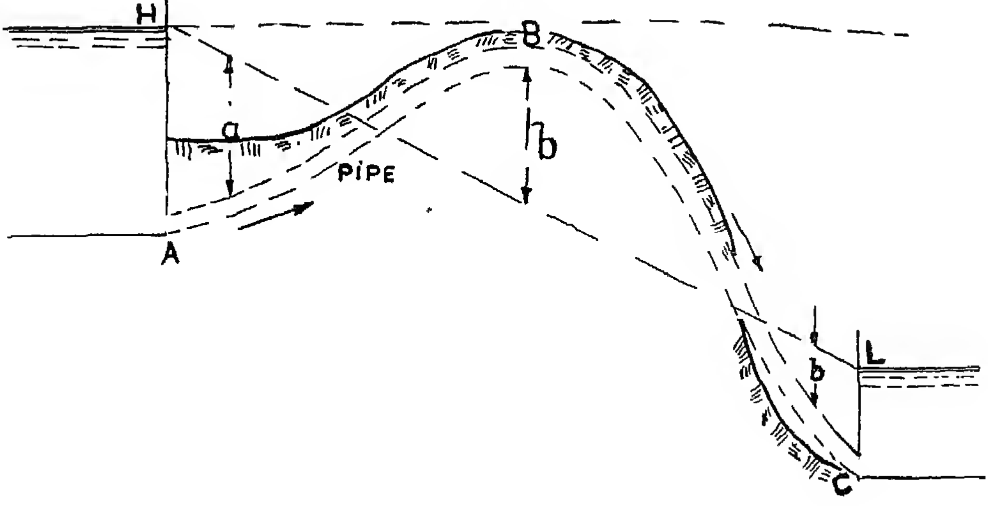
If the pipes take the course A_3 R^1 B_3 and if the water level in the reservoir Q is at point B_3, so that the pressure at B^3 is only atmospheric pressure, then the hydraulic gradient lines will be E R^1 and R^1 B_3 and not a straight line from point E to B^3.

This is the case with the gravitating main, for Simla Water Supply Scheme, from a small reservoir at Kufree town on Hindustan-Tibet road running along the ridge and hill side towards Simla.

In the above it would be seen that h feet is the loss of head in Length L feet. In half the length, it would be $\frac{h}{2}$ and at any point along the length L, it would be in proportion to the distance from the reservoir P. The corresponding point on the pipe line A B would be the point immediately vertically below the point on length L.

For town water supplies, the pipes are generally laid in very long lines and the Hydraulic gradient lines are only approximate, specially when the discharges vary so many times in a day.

Fig. 48 A



Let A B C be a pipe line, laid on an uneven ground surfaee, eonneeting two reservoirs H and L. H L is the Hydraulic gradient line, the water will lose energy at uniform rate while flowing.

Energy lost in overcoming frietional resistance is expressed in feet of water and is known as head lost in friction ; in the above sketeh, it is difference of level between H and L.

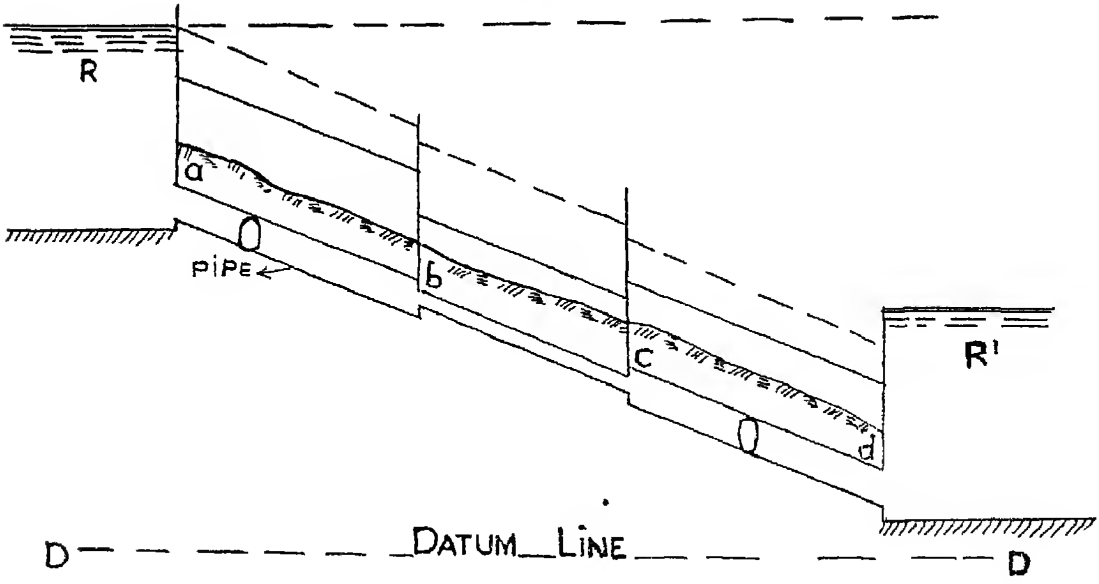
If P be the intensity of pressure of water at any section of the Pipe the pressure energy at that section will be $\frac{P}{W}$. HL is the pressure energy line.

Slope of Hydraulic gradient = differenee of level between points H and L, divided by the horizontal projection of the pipe line.

The vertical lines a and b represent the pressure energy in the pipe at those points and is above atmospheric pressure. The highest point of the pipe is B ; here the pressure energy is less than atmospheric and the least. If the absolute pressure at B is less than 8 feet of water, in other words 26 feet vaeuum, in other words point B is 26 feet above Hydraulic gradient line, the water commences to vaporise, and large bubbles will stop flow of water.

The pipe at B, though below the water level at H, will act partly as a syphon. Engineers should not lay their pipes more than 26 feet above the Hydraulic gradient line, preferably always below that line :

Fig. 48 B



In the above sketch, a b c d is a pipe line of varying section, connecting reservoirs R and R¹ : DD = a datum line. Mark at b, c, d, the losses of head due to all sources, to the same vertical scale as the figure ; at any point of the pipe line, the total energy of water will be datum head + velocity head and pressure head. The top points of ordinates at b, c, d, will lie on total energy line.

If V_1, V_2, V_3 are veloeities of flow in pipe lines ab, bc, cd, the following losses are to be taken into account.

At a, a loss due to entrance to pipe $= 0 \cdot 5 \dfrac{V_1^2}{2g}$ through the line ab, a uniform loss due to friction $= \dfrac{4flV_1^2}{2gd}$ (see formula 52A): f being ·01 approximately. At b, a loss due to sudden contraction $= \dfrac{\cdot 5 V_2^2}{2g}$; through the line bc, a uniform loss (due to friction). At C, a loss due to sudden enlargement $= \dfrac{(V_2 - V_3)^2}{2g}$. Along line c D, a uniform loss due to friction. At D, a loss due to velocity head being destroyed.

The sum of all three losses will equal the difference of level between the water surfaces in the reservoirs R and R¹.

If the velocity head is deducted from the total energy line, the Hydraulic gradient line will be obtained because it represents the pressure heads of water above the centre line of the pipe line.

Thus total energy above datum line — (velocity head) = Pressure head above centre line of pipe.

Velocity head between a and b $= \dfrac{V_1^2}{2g}$: b and c $= \dfrac{V_2^2}{2g}$: c and d $= \dfrac{V_3^2}{2g}$

If the diameter of pipes bc is much smaller than that of cd, the velocity V_2 will be greater than V_3. Hence the Hydraulic gradient of bc will be lower than that of cd.

70. *Coefficients.*—In article 67, equation 47 gives $V = C\sqrt{r}\sqrt{s}$. This formula is due to Chezy.* We will now consider the values of C. These for pipes are not well known. For smooth pipes the value of C is great. When the inside surface of the pipe gets incrusted due to heavy water or imperfect coating, the value of C is low. In a wooden pipe, C may be reduced by organic growth.

For town water-supply of Lahore, a 4″ C.I. pipe was found after thirty years' use, by the writer, incrusted to the extent of ¾″ all round, leaving the bore 2½″ for the discharge of the water.

Several experimenters Darcy, Smith, Fanning and others have found that value of C increases with both r and s. The following statement, due to Bellasis gives an abstract of mean coefficients found by Smith and Fanning for cast-iron pipes or riveted sheet iron or steel pipes, all supposed to be coated with antirust paint, joints smooth and curves of easy radii.

Taking the value of C from this table, V can be found.

Hydraulic Radius.	Hydraulic Gradients.			
Foot.	$\frac{1}{10}$	$\frac{1}{100}$	$\frac{1}{1000}$	$\frac{1}{10,000}$
1·0	..	143	135	131
0·25	120	113	104	100
0·10	107	101	96	,,

Kutter's coefficients, see article No. 97 are not very suitable for pipes; C remaining unaltered when 's' is increased above 1 in 1000. They give too low velocities for small pipes and too high velocities for large pipes with small slopes. However many engineers adopt Kutter's formula for calculating discharges through pipes.

* De Chezy, a French Mathematician of High Order.

If roughness of a galvanized iron pipe be taken as 1, that of other surfaces is approximately as follows:—

New uncoated cast-iron pipes............1·40: New asphalted C. I. Pipes 3·55: New Woodstave pipes 5·65: Concrete Pipe 1·50 to 6·00: Rough concrete pipes 18·50.

Southern Pacific Railroad Co. $V = c\sqrt{r}\ \sqrt{s}$.

Particulars of Pipes	Value of C.
8″ Plain pipe conveying oil	5·46
8″ Plain pipe conveying 90 per cent oil and 10 per cent water	7·11
8″ Rifled pipe conveying 90 per cent oil and 10 per cent water	65
8″ Pipe conveying water only	113
3″ Plain pipe conveying oil	3·76
3″ Rifled pipe conveying 90 per cent oil and 10 per cent water	79
3″ Rifled pipe conveying water only	100

f, in Formula 52A for Plain W. I. Pipes: Vel. 2 to 10: D 3″ to 60″ = ·00612 to ·00262.

71. *Discharge through pipes.*—For rough calculations for discharges through water mains, some engineers take C as 78. In that case

$$V = C\sqrt{r}\ \sqrt{s} = 78\sqrt{r}\ \sqrt{s} = 39\sqrt{d}\ \sqrt{s} \qquad \text{.......... 49.}$$

r = Hydraulic radius.

s = Sine of slope.

d = Diameter of Pipe.

The number of formulæ that have been proposed from time to time for calculating the flow of water in pipes is legion, but there seems, no doubt, that the formulæ of old investigators are not quite correct. It is generally recognised that probably the most accurate formulæ *hitherto* proposed are those of Fanning* in his exhaustive book on waterworks engineering.

Recently Professor Unwin has thoroughly gone into results of modern investigations by Fanning, Smith, Bonn, Lampe, Stearns as well as the older researches of Darcy and Chezy. Professor Unwin adopts the general formula:

$$\frac{h}{L} = \frac{m}{d^x}\ \frac{V^n}{2g}\left(\text{foot second-units}\right) \qquad \text{.......... 50.}$$

Where h is the head, L the length of the pipe, d the diameter of the pipe, V the velocity of flow. m, x, n, are numerical coefficients.

The coefficient 'm' depends on the nature of the surface of the pipe while 'x' and 'n' are exponents of the diameter and velocity respectively.

Professor Unwin gives the following values of the coefficients for foot-second units.

Kind of pipe.	m	x	n
Wrought iron	·0226	1·21	1·75
Asphalted iron	·0254	1·127	1·85
Riveted iron	·0260	1·39	1·87
New cast-iron	·0215	1·168	1·95
Incrusted cast-iron	· 044	1·16	2· 0

Formula for discharge through incrusted pipes from the values of coefficients given in the above table is as under:—

$$\frac{h}{L} = s = \frac{0\cdot044}{D^{1.16}}\ \frac{V^2}{2g} \qquad \text{.......... 51}$$

* An eminent Hydraulic Engineer of New York (1900).

A nomogram No. 3 showing discharges of incrusted pipes, prepared by C. L. T. Griffith on the basis of the above formula is given among the graphs attached to this book ; Fig. 61A. In the P. W. D. Handbook, Bombay, Vol. II, Eighth Edition (1931), on page 752, a formula by Hazen and William is given for discharge through pipes with C = 90 for incrusted pipes.

The formula is as under :—

$$V = C\, r^{0 \cdot 63} \times s^{0 \cdot 54} \times (\cdot 001)^{-\cdot 04} \qquad \ldots \ldots 52.$$

$$C = 90.$$

Tables showing discharges for different diameters of pipes for different values of s are also given in the said handbook. The results nearly agree with readings on nomogram No. 3, Fig. 61A, which has been worked out on the basis of Unwin's formula.

If by calculations, the diameter of a pipe for a given discharge and given slope, works out to 11 inches the nearest market size 12″ is to be adopted and so on. There is a limit to this also. While calculating discharges through pipes, the following limiting values of velocity are to be borne in mind.

Diameter in inches	..	4	8	12	15	24	36
Maximum velocity in feet per second	..	2·5	3·0	3·5	4·0	5·5	6·5

P. W. D. Handbook, Bombay Vol. II (1931), page 597 treats the flow of water in pipes as under :—

If V be the velocity in feet per second through pipe L Ft. long, the head h, absorbed in overcoming friction is, $h = 4\mu L V^2 \times \frac{1}{2gD} = \frac{\mu L}{R} \frac{V^2}{2g}$ (Darcy) ......52A.

R = mean hydraulic radius : D = Diameter of Pipe : μ = Coefficient of roughness of the inside surface of pipe,

$= \cdot 005 \left(1 + \frac{1}{12D} \right)$ for new well painted clean and smooth iron pipes.

$= \cdot 01 \left(1 + \frac{1}{12D} \right)$ for old incrusted pipes (Due to Darcy)

$= \cdot 0108$ or $\cdot 01$ nearly

$= \cdot 0026$ for varnished surface.

$$= V \sqrt{\left(\frac{2g}{\mu} \right)} \sqrt{\left(\frac{h}{L} \quad R \right)} = C \sqrt{R} \sqrt{s} \text{ where } C = \sqrt{\left(\frac{2g}{\mu} \right)}$$

For rough and trial calculations C, may be taken as 78 and $D = \cdot 2545 \sqrt{\left(\frac{Q^2}{s} \right)}$ ...52B.

H. M. Baines, Sanitary Engineer to the Government of Punjab (1910) patented a slide rule for calculating discharges through cast-iron pipes. The slide rule is based on Flamant's formula.

$\frac{h}{L} = K \frac{V^{1.75}}{d^{1.25}}$, K for C. I. pipes in service = ·000417.

For steel riveted mains 72″, 60″, 57″ several miles long for Bombay water-supply, the formula $V = C \sqrt{r} \sqrt{s}$ (Chezy), C being 108, was adopted under the advice of an expert (Prof. W. C. Unwin) who based this choice on the observations by Hershel and Marx for riveted steel pipes of large diameters*.

* See Proceedings of I. of E. (India), Vol. VII, December 1907. Tansa Water-works, Bombay.

72. *Short pipes.*—See article 22 and 25. In short pipes the head required to produce velocity and to overcome the contraction at entry must be taken into account. Article 68 deals with this part of the case. When the length of the pipe is not very great, the velocity may be high, the coefficient of discharge (C) may be outside the range of experimental data and its value then can only be estimated. For cases in which the length of the pipes is not more than 100* times d, the diameter of the pipe, the length may be treated as a short tube and velocity through the tube is $V = C\sqrt{2gh}$, C being taken from the article 25 as appropriate in the case.

The actual velocity may also be calculated as under :—

The velocity and therefore the discharge is proportional to $\sqrt{h}$. To find the velocity V in a given time, assume the velocity as V_1, estimate the heads required (*i*) to produce V_1 and (*ii*) to overcome resistance at entry and sum these heads, then

$$\frac{V}{V_1} = \sqrt{\left(\frac{\text{Actual head}}{\text{Estimated heads}}\right)} \quad \ldots\ldots 53.$$

Example :—Find the discharge of a 12 inch pipe, 15 ft. long with a head of 4 feet, assuming the velocity to be 10 feet per second.

See equation 49.

$V = 39\sqrt{ds}$ $\therefore$ $V^2 = 39^2$ ds. $V_1 = 10$ ft. per second assumed.

$10^2 = 39^2$ ds. : Length of pipe = 15 feet : $\therefore s = \frac{h}{15}$.

$10^2 = 39^2 \times 1 \times \frac{h}{15}$ $\therefore h = 0{\cdot}99$ ft., head required to produce V_1, velocity to overcome resistance due to friction in pipes. Let h_1 be the head to overcome resistance at entry and to generate the velocity at start.

$h_1 = (1 + \alpha)\frac{V^2}{2g}$, α being $0{\cdot}5$: $V = V_1$ assumed.
$= 10$ ft./sec.

$h_1 = 1{\cdot}5\,\frac{V_1^2}{2g}$ $\therefore h_1 = 2{\cdot}34$ feet.

$h + h_1 = 0{\cdot}99 + 2{\cdot}34 = 3{\cdot}33$ feet, total head for velocity 10 feet per second.

We have seen above (equation 53) that $\frac{V}{V_1} = \left\{\frac{\text{Actual head}}{\text{Estimated head}}\right\}^{\frac{1}{2}}$

$\therefore$ V the actual velocity $= 10\sqrt{\frac{4}{3{\cdot}33}} = 10{\cdot}95$ ft. per second.

Discharge Q = area of the pipe $\times v = \frac{22}{7} \times \frac{1}{4} \times 10{\cdot}95 = 8{\cdot}6$ cubic feet per second.

If the entry is bell-mouthed, the coefficient of discharge for the orifice may be as high as $0{\cdot}97$, so that $h_1 = 1{\cdot}06\,\frac{V^2}{2g}$

If h be the head required to maintain V actual velocity, then $h = \frac{\mu\, 4LV^2}{d2g}$ (equation 52A : also P. W. D. Handbook, Bombay, page 595) : If h_1 be the head required to overcome resistance at entry and generate velocity at start, then $h_1 = 1{\cdot}5\,\frac{V^2}{2g}$ for cylindrical entry as given above.

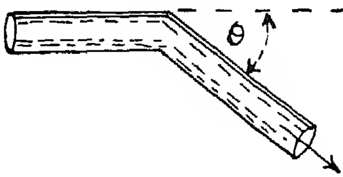
$$V = \left(\frac{2gHd}{1{\cdot}5d + 4\mu L}\right)^{\frac{1}{2}} \qquad \text{Bellasis} \quad \ldots\ldots 54.$$

d = diameter of Pipe : $\mu = {\cdot}01\left(1 + \frac{1}{12}d\right)$ for old pipes.

$H = h + h_1$: L = Length : $\mu = {\cdot}005\left(1 + \frac{1}{12}d\right)$ for new pipes.

* Head of Water 100 ft. : Diameter of Pipe 1 ft. : Length of Pipe 100 ft. : Vel. = 43 F/S : 1,000 ft., V = 14 F/S : 10,000 ft., V = 4·4 F/S.

Fig. 49



73. *Loss of head by Elbows, Bends, etc.:*—See Bombay P.W.D. Handbook, page 595.

(*i*) *Elbows.*—Loss of head $= \left(\frac{1}{2} \sin \theta^2 \right) \frac{V^2}{2g}$

The loss is due to contraction in the stream, as shown in Fig. 49.

(*ii*) *Bends.*—Loss of head due to a bend is given by the following Weisbach's formula.

$$h = \left\{ 0{\cdot}13 - 1{\cdot}85 \left(\frac{d}{2p} \right)^{\frac{7}{2}} \right\} \frac{V^2}{2g}$$, d being diameter of pipe and p being radius of bend....55.

(See Table, Bombay P.W.D. Handbook, Page 596).

(See also page 367 of Molesworth's Pocket-book, 29th edition, 1927 and proceedings of the American Society of Civil Engineers, Vol. xxvii, page 314).

In fact the resistance in bends is not well known. For purposes of calculations, it is generally sufficient to simply add, say 2 per cent to the total head, to allow for losses for bends and other contingencies and neglect the detailed calculations for resistances in bends.

(*iii*) *Loss of head in enlargements:*—If V and V_1 be the velocities in the small and large portions of the pipe, the loss of head $h = \frac{(V-V_1)^2}{2g}$.

If d and d_1 be the diameters of smaller and larger portions of the pipe, loss of head

$$h = \frac{V_1^2}{2g} \left\{ \left(\frac{d_1}{d} \right)^2 - 1 \right\}^2 \qquad \text{.......56.}$$

See Fig. 14: article 12.

IV. *Loss of head in contractions.*—This is equal to $\frac{V^2}{2g} \left(\frac{1}{C_c} - 1 \right)^2$ ....57.

Where C_c = Coefficient of contraction = 0·6 in this case.

$h = 0{\cdot}44 \frac{V^2}{2g}$. In long pipes when the length of the pipe line is greater than 1,000 diameters, the loss of head at entry, being a small quantity, may be leftout for simplifications. The error in computing velocity in long lines does not exceed 5 per cent. At point of change in the diameter of a pipe, taper pieces are used, and at angles in pipe lines easy bends of curvature of large radii are adopted.

V. *Parallel flow through pipes. Two pipes branch off from point A:*—

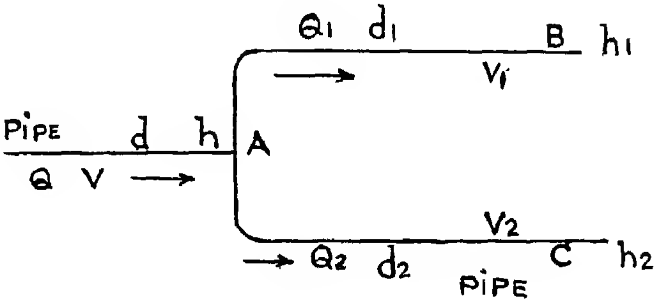
The following principles hold good.

$Q = Q_1 + Q_2$ or $d^2 \times V = d_1^2 V_1 + d_2^2 V_2$

Head $h\text{-}h^1$ = head absorbed in friction through pipe d_1

$h\text{-}h_2$ = head absorbed in friction through pipe d_2.

Fig. 49 A



While designing a new pipe system, if the quantities Q_1 and Q_2 vary, then design the diameters d_1 and d_2 in such a way that the pressure head h_1 and h_2 are equal nearly.

If the pipe system is already in service and full of water, then fix temporarily hydraulic pressure gauges at points B and C, and ascertain h_1 and h_2 and also h ; thus find V_1 and V_2 and discharges.

74. *Inclination and combination of pipes.*—In practice pipes must follow the surface slope of the ground in which they are laid and must therefore be in segments laid at various slopes.

Suppose

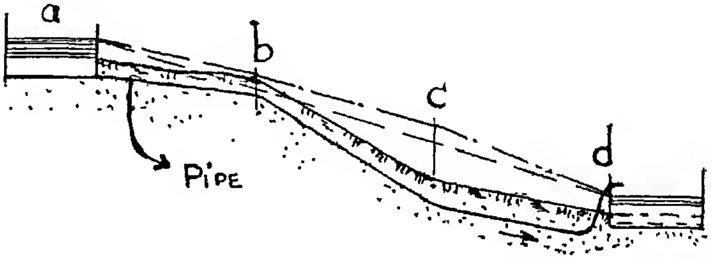
(*i*) The diameter of the pipe is uniform throughout and a given discharge is required at the end of the pipe and also this discharge is uniform throughout the pipe. In this case the virtual slope is practically a straight line whatever be the slopes of the different segments of the pipe line.

$$\text{The discharge } Q = A \times V = \frac{\pi d^2}{4} \times C \sqrt{\frac{d}{4}} \sqrt{s} = \frac{\pi}{8} c\, d^{\frac{5}{2}} s^{\frac{1}{2}}.$$

If L be the total length of the pipe line, and H the total head lost, then $S = \dfrac{H}{L}$. It will be seen that Q varies as $d^{\frac{5}{2}} s^{\frac{1}{2}}$:

$$\therefore s^{\frac{1}{2}} \text{ varies as } \frac{1}{d^{\frac{5}{2}}} \text{ or s varies as } \frac{1}{d^5} : \text{or } \frac{H}{L} \text{ varies as } \frac{1}{d^5} \text{ or H varies as } \frac{L}{d^5} \quad \ldots\ldots 58.$$

Fig. 50



(*ii*) If the course of a pipe line is fixed and the discharge is also fixed, the pipe line may be divided into segments ab, bc, and cd, of different lengths and different slopes, the pipes will be laid about 3 feet below the ground surface, as shown in the figure No. 50. If the hydraulic gradient line be assumed one line from a to d and sizes of pipes be calculated accordingly, there will be an air-lock at point b as air will collect at b and the pipe will not run full, because the pipe at b will be above the supposed hydraulic gradient line ad. This is defective. Since the discharges and slopes of ground surface for segments, ab, bc, cd are given, calculate the diameters of pipes and mark the hydraulic grades ab, bc, cd as shown by " dash and a point " dotted lines in the figure 50. Thus the hydraulic grade line will start from a to b, thence b to c and thence c to d, ending where the segment of the pipe line ends. This is the proper way.

When the diameter of the pipe ab is fixed for discharge and for slope of the ground surface ab, the diameter of the lines bc and cd can be found out because s varies as $\dfrac{1}{d^5}$

$$\therefore \frac{S_1}{s} = \frac{d^5}{d^5_1} = \left(\frac{d}{d_1}\right)^5 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 59.$$

If the vertical difference between points b and c is very great and there is a danger of the pipe line being burst due to abnormal pressure, then introduce break-pressure valves or break-pressure tanks.

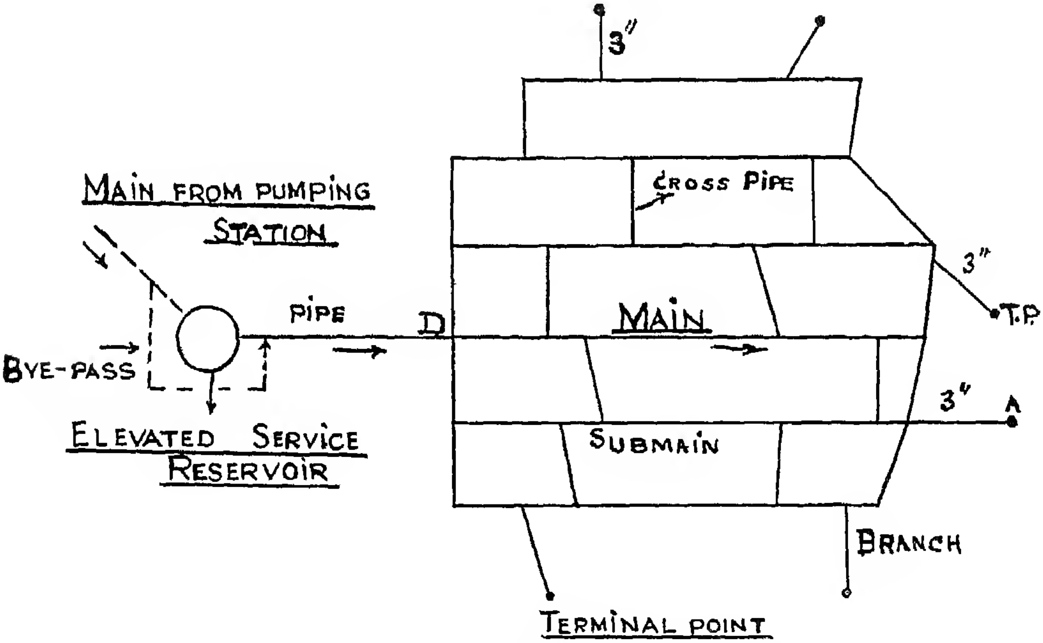
For Simla water supply such tanks were designed on the gravitating main from Kufree service tank to the Simla town by the writer in 1910.

Generally there is a sluice valve at point a to regulate the supply of the water to the gravitating main. When the sluice valve is fully closed, the water passes down the pipe ab, and there is a negative pressure (partial vacuum) inside the pipe just below the sluice valve. It is likely that the pipe may burst at that point. Therefore an air valve is absolutely necessary just below the sluice valve above point b.

(*iii*) There is a third case when the conditions are similar to those of case No. *ii*, with the exception that the discharge varies from point to point due to branches taking off from the main pipe at different points and again sub-branches taking off from branches to supply water to house services. This is the case of a water-supply distribution system for a town. Indeed this is a very complicated problem. The terminal head at the end of a sub-branch near a building in a large town is generally 40 to 50 feet to protect the buildings against fire. If the buildings are only one storeyed, then the terminal head is about 30 to 40 feet, varying with the heights of buildings in the town or the city.

Fig. 51 shows a distribution system of cast iron pipes for an ordinary town. Water is pumped in the elevated reservoir, built on a hill or on a high staging.

Fig. 51



The population of the town is known by districts or wards and is determined for each area served by street branch and for all sub-mains and mains. The allowance for water-supply per each person per day varies from 10 gallons to 30 and 40 and sometimes 50 and more, depending upon the importance of the town or the city.

Suppose the ground surface level at the farthest terminal point is 100·00. The terminal head proposed, say is 40 feet. Then the highest level to which the water should rise at the terminal point is 140. The water level in the elevated reservoir is say 180. So we have 40 feet head available for overcoming resistance in the chief gravitating main and branches right up to terminal point A. We also know discharges which this main should carry from junction point to junction, upto the point D.

The main from point D to the elevated tank is to carry supply for the whole town. Thus calculate the sizes of the pipes from point to point according to the required discharges, keeping in mind that 40 feet only is the available head to be distributed from point to point economically.

The above is the usual way to work out the distribution system. The writer had had experience of designing and examining several water-supply distribution systems. It was found that in practice the system does not work fully as designed. The water taps on or near the chief gravitating main draw more water due to high pressure than their allotted quotas, and the taps furthest from the point D, into the streets supply less water than anticipated. Sometimes, the taps due to high pressure discharge water in abnormal quantities and empty the service reservoir too soon (Ahmedabad city water-supply distribution system 1931).

The remedy lies in designing terminal branches never less than 3 inches to 4 inches in diameters.

Two inch cast iron mains were used in terminal streets at Iskhel in Punjab, resulting in poor or no supply to the people in the street.

At Ahmedabad, to supply water on upper storeys of houses in the city, an elevated tank was built in Rambaugh (1930). The tank was built so high up in the air that the pressure at the stand-posts in the streets was so great that the abnormal discharge of water from the taps emptied the tank within 3 hours in the morning and there was no supply to the people after 9-0 a.m. The President of the Municipality advertised for advices from engineers in the market to remedy the bad state of affairs in the distribution system.

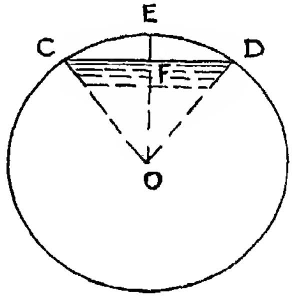
It will be seen that discharges of water in mains in towns form a very difficult problem. When the pipes exist in streets and carry water-supplies, hydraulic pressure gauges should be put up at different points and pressures read at different times during supply hours. But if the distribution system is to be newly designed, experienced engineers should be consulted.

Allowance for incrustation of pipes should always be made, say one inch addition to the diameter calculated.

For pumping mains of long lengths, additional investment for a large pipe should be balanced against the extra cost of pumping through a small pipe when incrusted. In such a case, the rate of interest expected and the cost of pump horse-power per year, actually determine the case.

Velocities should be kept under 4 feet per second to avoid bursting of mains due to water hammer.

Fig. 52

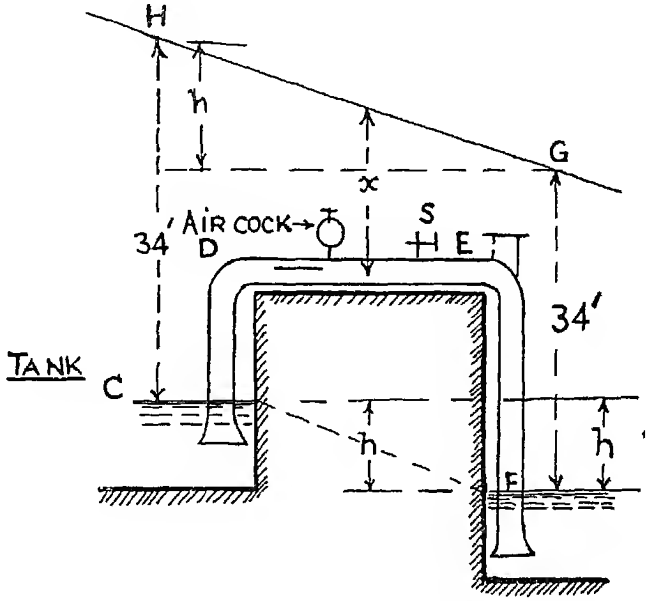


75. *Pipes not running full.*—When the pipe lies on ground and there is gravity flow in the pipe, which is not under any pressure head, the pipe is said to lie on its own virtual slope or the hydraulic gradient has the same inclination as the ground on which the pipe is laid.

In this case, the Hydraulic mean radius is not $\frac{d}{4}$ but is $\frac{A}{B}$, A being the area of water section and B the length of wetted arc.

The discharge Q varies as $\left(\frac{A}{B}\right)^{\frac{1}{2}}$ and is maximum when the angle C O D is 54 degrees.

Fig. 53



76. *Syphon.*—In its simplest form a syphon consists of an inverted U tube, both legs being full of water as shown in fig. 53.

Suppose a large tank has to be emptied or water is to be drawn from a canal where no outlet sluices exist. In that case a syphon is put up across the bank as shown in the figure. When the syphon works, a column of water enters the leg CD, travels along DE and falls down the tube EF, creates vacuum and the flow becomes continuous.

The difference of level between points C and F is h. The atmospheric pressure at C and D is the same, equal to height in water barometer 34 feet. When vacuum is created in the syphon, it is the atmospheric pressure that forces up water in the leg CD. Set-off CH and FG, each 34 feet to represent the atmospheric pressures. HG is the imaginary Hydraulic gradient line and h is the difference of level between the two points H and G, equal to the difference between C and F.

To start the syphon, the tube CDEF must be filled with water through an inlet at E and closing the outlet at F temporarily. If this is not possible then use a suction pump and suck out air from the syphon through the suction cock at S. As soon as vacuum is wholly or partly created in the syphon tube, water rushes in the tube at C and the syphon comes into play. If the crest of the syphon DE is so placed or is so high that it lies above the line HG, which means if the legs CD and EF are abnormally long, there will be incomplete vacuum in the crest DE and the syphon will not work.

The vacuum to be created inside the syphon is measured by the quantity $34 - x$ feet. The greater the value of x, less the vacuum to be created, less the height of the leg CD, the syphon will then work easier.

When in action, the air disengaged from flowing water tends to collect in the bend DE, an aircock with a small air vessel should be provided in order that full discharge may be maintained.

For calculating discharge through the syphon, the tube CDEF, should be treated as a short pipe working under a total head of h feet: See article 72 and the formula therein including the equation 54.

First there is a loss of head at entry by shock and generation of velocity, equal to $1 \cdot 5 \frac{V^2}{2g}$ for cylindric entry. Second item is the loss of head in bends. Third item is the loss of head by friction in the whole pipe of length L. This is equal to $\frac{4fLV^2}{2gd}$ (equation 52A).

$$V = \left(\frac{2gHd}{1 \cdot 5\,d + 4\,fL}\right)^{\frac{1}{2}} \left(\text{See equation 54}\right).$$

The sum of all these three losses of head should not exceed the available head 'h' as shown in fig. 53.

The above calculations can approximately be solved by equation 49 article 70, whose S here equals $\frac{h}{L}$, L being the overall length of the syphon tube.

Due to air particles being contained in water, the water barometer reading 34 feet is generally taken as 28 feet in calculations.

In the fig. 53, it is evident that shorter the leg CD and the smaller the length of the crest DE, less the amount of vacuum to be created. The crest DE should not be so long as to touch the imaginary Hydraulic-grade line HG.

There is a case when the water-level in the tank is just at the top of the weir and is to be retained as maximum, any further rise in the water-level will cause surplus discharge into the pipe DEF, which will function as syphon spill away with a greater discharge for a given length than an ordinary open weir because we utilize this whole difference of head between points E and F. The discharge then $Q = C\,\alpha\,\sqrt{2gh^1}$: α being the area of the syphon pipe and h^1 being the height EF, C being the coefficient of discharge to be determined by test. Such spillway syphons can be of different sizes and fixed at different heights of the Dam and serve as substitutes for the weir.

For Jamshedpur water-supply, a syphon spillway has been put up at the end of a storage dam to dispose of surplus water during heavy rains. This reduces the flood lift and increases the storage. This particular design gives a high coefficient of discharge.

Sometimes as at Indore, a baby syphon is put up to prime the main syphon easily.

Example :—Estimate the discharge of a syphon 3′-6″ in diameter and 240 feet long, the difference of water-levels being 12 feet.

Assume the velocity to be 10 feet per second. By equation 49, article 70,

$V = 39\sqrt{ds}$ $\quad d = 3\frac{1}{2}$ feet $\quad s = \frac{h}{240}$, h being the head lost to overcome friction in the syphon.

$$\text{Then } 10^2 = 39^2 \times 3\tfrac{1}{2} \times \frac{h}{240}$$

$$h = \frac{100 \times 240 \times 2}{1521 \times 7} = 4 \cdot 51 \text{ feet.}$$

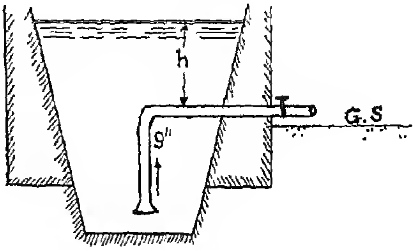
$$h_1 = \text{head lost at entry} = \frac{1 \cdot 5V^2}{2g} = \frac{3}{2} \times \frac{100}{64} = 2 \cdot 34 \text{ ft.}$$

Total estimated head $h + h_1 = 6 \cdot 85$ feet but actual head is 12 feet.

See equation 53. $\therefore$ actual velocity $= 10\,\frac{\sqrt{12}}{\sqrt{6\,85}} = 13 \cdot 23$ feet per second.

Discharge $= A \times V = \frac{\pi}{4}\,(3\frac{1}{2})^2 \times 13 \cdot 23 = 127$ cusecs. For more correct solution, apply equation 54.

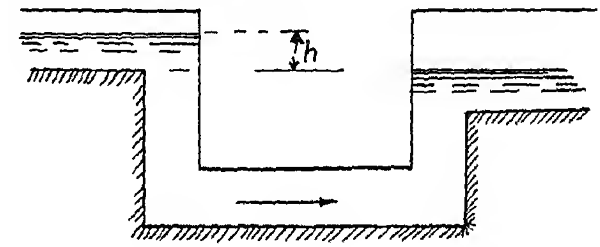
Fig. 54



77. *Semi-syphon*.—Fig. 54 shows an arrangement, put up by the writer at Kilokiee sewage pumping station to take out sludge from a settling tank. A 9" cast-iron pipe working under a head h feet (about 6 feet) discharges out the sludge from the bottom of the settling tank when the sluice value outside the tank is opened.

Thus the settling tank is kept clear of the sludge by opening the sluice valve once or twice a day.

Fig. 55



78. *Inverted syphon.*—The loss of head through the barrel of a syphon, where there is no velocity of approach, may be calculated by the formula,

$$h = \left(1 + f_1 + f_2 \frac{1}{r}\right) \frac{V^2}{2g} \quad \ldots\ldots 60.$$

(Bombay P. W. D. Handbook, vol. II, page 597).

Coefficient f_1 may be taken as 0·8 for a bell-mouthed syphon and 0·505 for a cylindrical mouth

Coefficient f_2 is $= a\left(1 + \frac{b}{r}\right)$ where the value of a and b are to be taken as follows :—

Inner surface of barrel	*a*	*b*
Smooth iron pipes	·00497	·021
Incrusted iron pipes	·00996	·021
Smooth cement or planed timber	·00316	0· 10
Ashlar, brickwork or planks	·00401	0· 23
Rubble masonry or stone pitching	·00507	0· 82

The mean velocity in the barrel when the head (h) is known, is :—

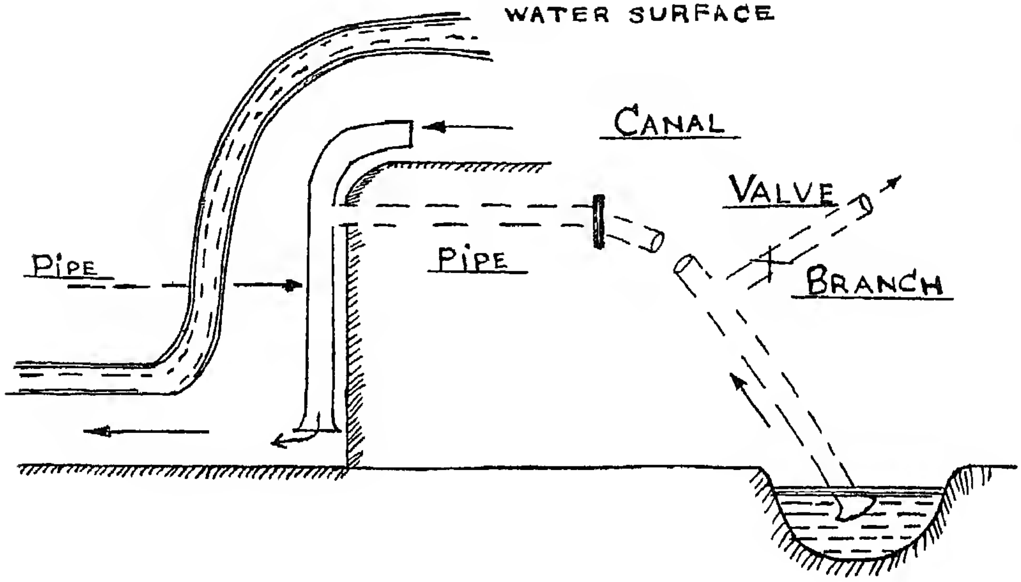
$$V = \sqrt{2g}\,\sqrt{\left(\frac{h}{1 + f_1 + f_2 \frac{1}{r}}\right)} = 8{\cdot}025\,\sqrt{\left(\frac{h}{1 + f_1 + f_2 \frac{1}{r}}\right)} \quad \ldots\ldots 61.$$

If there be velocity of approach, it should be neglected to compensate for loss of head due to bends, etc.

When a canal has to cross another canal or a railway line or a road, the above arrangement as shown in fig. 55 is generally adopted.

For constructional details, the engineer is advised to refer to detailed drawings in record with P.W.D. Secretariat Offices of different provinces in India.

Fig. 55A



79. *Hydrostat.*—Fig. 55A shows an arrangement on Chenab canal in Punjab for lifting rain water from low lands and delivering the same to high lands for irrigating crops or watering fruit trees.

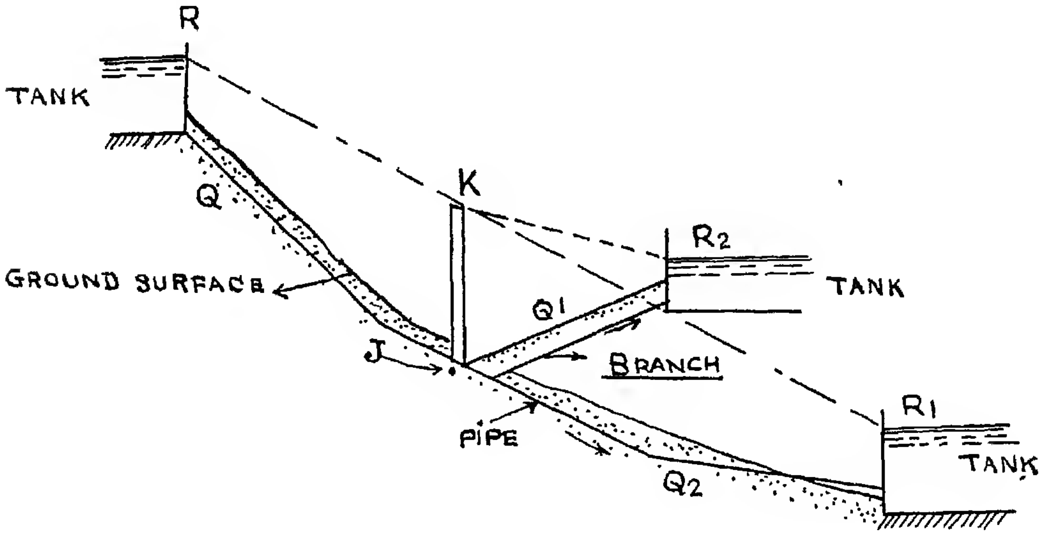
There is a fall in the bed of the canal. An iron pipe is fixed at one end of the fall and water allowed to fall through this pipe. A pipe of smaller diameter connects this main pipe and the pool of water in the low land. As the water falls through the main pipe, Torricelli's* vacuum is created in the small pipe and water, due to atmospheric pressure, rushes into the small pipe. If a valve on the branch pipe, leading to cultivated lands, is opened, the water flows on to the land, provided the difference of level between the valve and the water surface in the pond is less than 34 feet.

For further details, reference to be made to Punjab P. W. D. Secretariat, Irrigation Branch, Lahore.

80. *Branched main supplying two or more service reservoirs.*—Case 1. Uniform main with uniform discharge. Suppose two reservoirs R and R^1 exist and a water main of uniform diameter carries water (quantity Q) from R to R^1. It is proposed to build a reservoir at point R_2 and feed it with water (quantity Q_1), by a branch taking off at point J from main pipe, leaving quantity Q_2 for reservoir R^1, Q_2 being $Q - Q_1$.

* Torricelli, an italian evangelist (1608-1647): Galileo's pupil at Florence: invented Barometer in 1643: Improved Telescope and Microscope.

Fig. 56



When the water main is carrying quantity Q, note the pressure at J by a hydraulic gauge and plot it by a vertical line JK. The pressure gauge reading should equal the height JK arrived at by calculation. RK is the hydraulic grade line for discharge Q through main RJ. KR_2 is the hydraulic grade line for discharge Q_1 through the branch; as Q_1 is known the diameter of the branch can easily be ascertained *vice versa*. The point R_2 must be at a lower level than the point K.

The lower part JR_1 is of the same diameter as the upper part RJ of the main pipe line. It must now discharge only $Q - Q_1$ quantity and not Q as before, under the same conditions.

To achieve this, fix a sluice valve on the main line just below the point J and a sluice valve on the branch at point J, to control discharges to R_1 and R_2, which can also be gauged in the reservoirs.

Case II.—A main bifurcating into two branches.

In the fig. 56, RJ is the proposed main and JR_1 and JR_2 are the proposed branches. The proposed discharges are Q and Q_1 and Q_2 as shown in figure 56. $Q = Q_1 + Q_2$: The diameters of the pipe line are to be calculated.

Assume the diameter of the line RJ and calculate the hydraulic grade line RK. The point K must be higher than the points R_2 and R_1. Otherwise assume another diameter for the line RJ When the reduced level of the point K is approximately known, then the hydraulic grade KR_2 and KR_1 are known; from these calculate the diameters of the lines JR_2 and JR_1.

The three diameters should be sensibly proportionate, otherwise assume a different diameter for RJ and by such trials, the problem may be solved. In article 74, the comparative relationship of hydraulic grades and diameters of pipe are given. These will be useful while doing the above calculations.

81. *Jet and a Nozzle :—*

Fig. 57

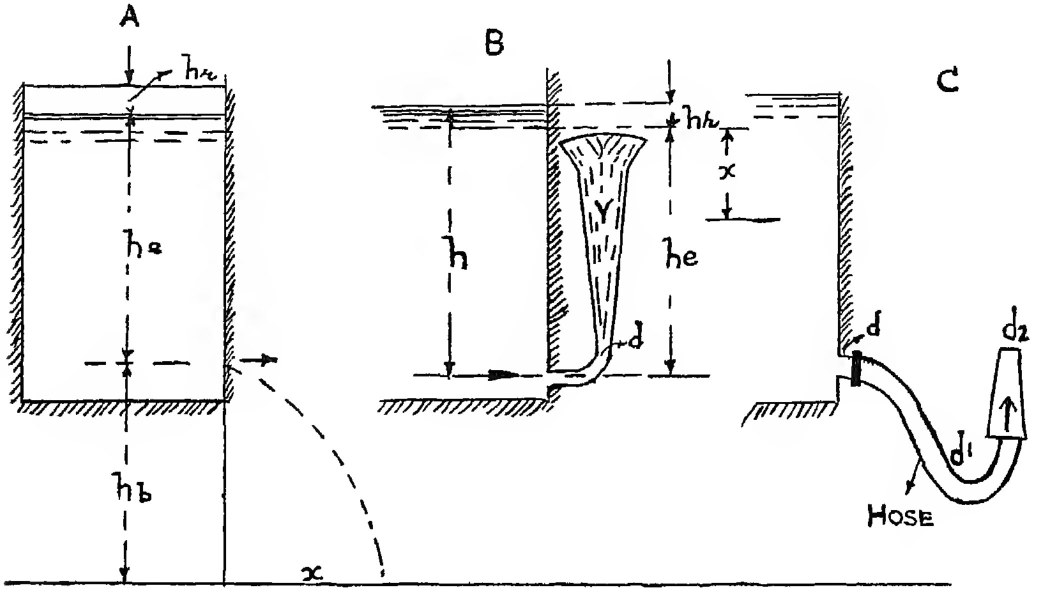


Fig. 57-*B* :

h = actual head : he = effective head. h — he = hr = head wasted in overcoming resistance

$\frac{hr}{he}$ = Cr = coefficient of resistance : C_V = coef. of vel.

$$V = C_V \sqrt{2gh} = \sqrt{2ghe} \quad \ldots\ldots\ldots\ldots 62.$$

V being actual velocity : C_V being coefficient of velocity, varying from 0·995 to 0·70. d = diameter of jet.

The velocity of water threads issuing in a jet, is as if the particle of water fell through height h, friction being neglected. Jet is formed because the water issues from a small orifice under pressure, as in the case of an ornamental fountain or a fire engine.

In order that a jet supplied by a pipe may ascend to the greatest height, the orifice should be fitted with a piece of hose, having a nozzle at the other end. The nozzle is a device invented to convert the total head of the water into velocity head. It is also used in some form of turbines. The pressure of the jet issuing from the nozzle is atmospheric and therefore the whole of energy is Kinetic. The loss of head in the nozzle is very small and therefore neglected.

The nozzle should be a conical convergent piece of pipe because it gives a high coefficient of velocity. The diameter of the hose pipe (fig. 57C) should be 3 times d, the diameter of the orifice. The actual height to which the jet rises, he $= \frac{V^2}{2g}$. Allowing for correction due to resistance of the air he = he (1-·003 he^2) (weisbach). If we take a point on the jet, at a distance of x feet from the top of the jet, and y be the diameter of the jet at that point, then

$$Y = d \left(\frac{he}{x}\right)^{\frac{1}{4}} \quad \ldots\ldots\ldots\ldots 63.$$

The velocity of the jet decreases uniformly. When the head h is great, the jet does not retain its coherence long enough to rise to the height he.

If the jet issues horizontally (fig. 57A), the range x on a horizontal plane $= 2\sqrt{he\ hb}$ .... 64.

x is also $= Vt : hb = \frac{1}{2} gt^2 : V = \left(\frac{gx^2}{2hb}\right)^{\frac{1}{2}}$

x is maximum when $he = hb$:

$hb = \frac{(\frac{1}{2}x)^2}{he}$ V = velocity of efflux at the orifice.

Fig. 57(C) shows a smooth conical nozzle, attached to a hosepipe. The length of the nozzle is generally 6 to 10 times d, the diameter of orifice. The pressure p at the entrance to the nozzle being measured by a pressure gauge, the head on the nozzle is $\frac{P}{w}$. The coefficient of velocity for smooth nozzles varying from $\frac{3}{4}''$ to $1\frac{1}{4}''$ in diameter, is 0·98 for pressures from 15 to 80 lbs. per square inch.

In large cities fire-engines are sometimes seen and the jet goes right upto the top of the building to extinguish the fire. The following table gives the total heights attained by a jet under different pressures.

Pressure in pounds per square inch.	Pressure head in feet.	Smooth nozzles. 1″	$1\frac{1}{4}''$	$\frac{3}{4}''$
10	23	22	23	..
20	46	43	43	..
30	49	62	63	59
50	115	94	99	92
70	161	121	129	113
100	230	148	164	133

The total height to which the jet remains serviceable as a fire stream is less than that to which the scattered drops rise by 20 per cent.

In fire-engines, the pressure head is created by hydraulic pumps worked by hand or steam, to carry the jet to the required height.

In hill stations, to utilize the falls in hill streams, flour mills have been put up from ancient times and the ancients were wise to use conical nozzles to create high velocity in the jet impinging upon the blades of the driving wheel, working the millstone.

The horse-power of a jet of water is as under :—

α = Area of cross section of the jet in square feet, (at outlet end of the nozzle).

V = Velocity of jet in feet per second.

Q = discharge $= \alpha \times v$ Cusecs : ω = weight of 1 c. ft. of water.

W = weight of discharged quantity $= \omega\ \alpha\ v$ Lbs.

Kinetic energy $= W \times h$ foot-pounds $= W \frac{V^2}{2g}$ ft.-pounds per second.

$= \frac{WV^2}{2g} \times 60 = \frac{\omega\alpha V^3}{2g} \times 60$ ft.-pounds per minute.

Horse-power $= \frac{\omega\alpha V^3}{2g} \times \frac{60}{33000} = \frac{\omega\alpha V^3}{2g \times 550}$ .... 65.

See fig. 57. The height $he = \frac{V^2}{2g} = \frac{h}{1 + \mu \frac{4L}{d_1}\left(\frac{d_2}{d_1}\right)^4}$ .... 65A.

L = being the length of the hose or connecting pipe : d_2 = diameter of the nozzle : d_1 = diameter of the hose.

82. *Water hammer or hammer-blow in pipes.*—If a valve on long pipe in which water is flowing with velocity V feet per second, is closed suddenly, pressure suddenly increases because the energy of moving water is suddenly checked, and a pressure wave travels along the pipe, setting up Noises known as knocking. The intensity of pressure will depend upon the speed at which the valve is closed and on length of the pipe.

This sudden jump in the pressure in the pipe due to the stoppage of flow is known as the hammer-blow.

Let the pipe be L feet long, cross sectional area $= \alpha$ sq. feet. Velocity of flow V feet per second. $t =$ time in seconds in which the flowing water is stopped by closing of the valve.

Then retardation of water, $f = \frac{V}{L}$ ft. per second. Weight of moving column of water $= \text{mass} \times g = \alpha L \times \omega$:

$\therefore$ Mass $= \frac{\omega \alpha L}{g}$: $\omega =$ weight of one cubic-foot of water.

Now force = mass $\times$ retardation.

Force on valve $= \frac{\omega \alpha L}{g} \times f$. lbs.: Intensity of pressure on valve, $P = \frac{\text{force}}{\text{area}} = \frac{\omega L f}{g}$ lbs. per sq. ft.

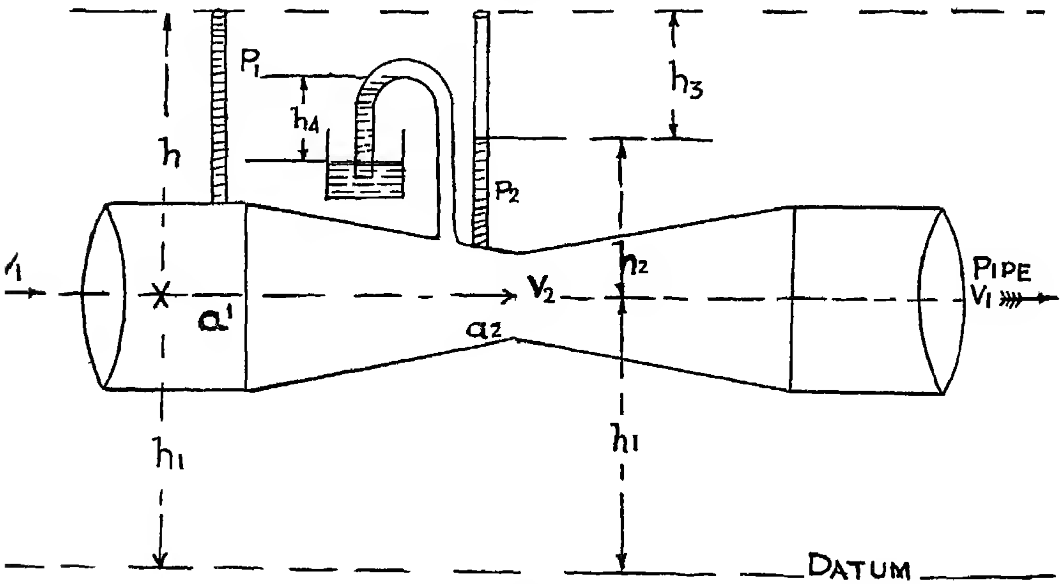
$$P = \frac{\omega L V}{gt} \text{ lbs. per sq. ft.} \qquad \ldots\ldots 66.$$

From the above equation magnitude of the pressure wave can be found.

Hydraulic drills used in mines and tunnels are worked on the above principle of wave transmission. A blow from piston is given to the water in a long pipe, which causes a pressure wave to travel the full length. At the other end of the pipe, the pressure wave works the drill.

Thus the water does not move along the pipe but the power is transmitted.

Fig. 58



83. *Venturi meter.*—When a quantity of water passes through a pipe, it is measured by a Venturi meter, the apparatus being more costly than a weir but very accurate and entailing practically no loss of head. It was used by Clements Herschell. It consists of two cones joined at small ends, forming a constriction in a pipe as shown in Fig. 58. Point a_2 is called the throat of the meter. P_1 and P_2 are the pressure pipes inserted in the pipe line to measure pressure. V_1 and a_1 are velocity and area of the main pipe. V_2 and a_2 are velocity and the area of the throat. The discharge through the pipe line is Q and constant $\therefore$ $V_1 a_1 = V_2 a_2 = Q$.

The loss of head due to friction etc. on account of flow of water in this short length of pipe is so small that we neglect it altogether.

By Bernouli's theorem, the kinetic energy of a particle of water moving along the centre line of the pipe line is the same at every point, above a given datum. $h_3 = h - h_2$ (pressures in tubes P_1 and P_2) $\therefore h_1 + h + \frac{V^2}{2g} = h_1 + h_2 + \frac{V^2_2}{2g}$: also $V_1 a_1 = V_2 a_2 = Q$ as given above.

Interchanging the different values, $Q = \left\{ \frac{a_1 a_2}{(a_1^2 - a_2^2)^2} \sqrt{2g} \right\} \sqrt{h_3}$ ....67.

The figures within the larger brackets are constant for a given meter and may be denoted by a constant equal to C.

It is also usual to allow for friction and shock by the coefficient K, found by experiment, thus $Q = KC(h_3)^{\frac{1}{2}}$ ....68.

K being 0·97 on an average. The head h_3 actually measured is called the Venturi head.

It will be seen that the diameter of the throat being reduced, the velocity increases to maintain the discharge Q ; therefore the pressure at the throat decreases. If this pressure P_2 falls below the atmospheric pressure π and a hole be drilled at the throat, no water will come out but air will enter the pipe.

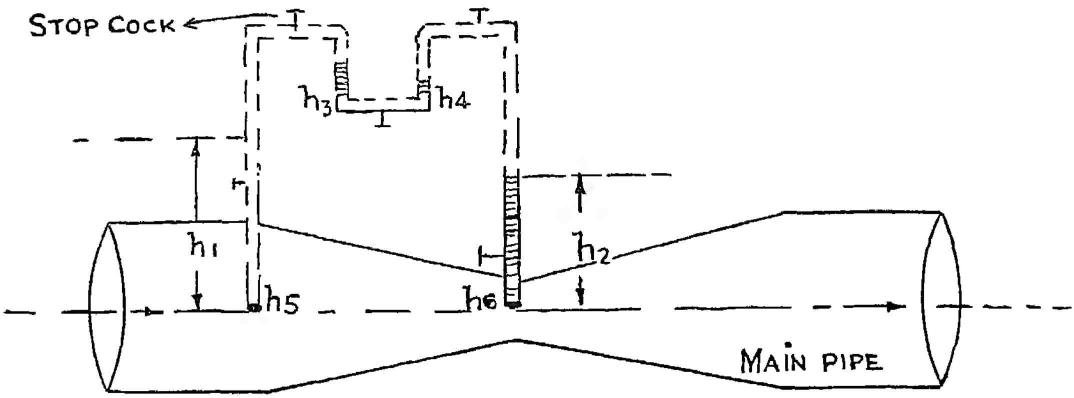
The height h_4 measures the difference of pressure between π and the pressure inside the throat, if a special tube is set up, as shown in the sketch, at the throat.

There is a limit to the ratio of the diameter of main pipe to that of the throat and this is reached when the throat pressure is nearly 8 feet of water absolute ; because at this pressure water evaporates and will hinder the flow in the throat.

Herschell gives standard proportions for the Venturi meter. If d be the diameter of the main pipe, then the diameter of the throat should be $\frac{d}{3}$ and the length of the receiving cone $= 2 \cdot 5d$ and the length of the discharge cone $= 7 \cdot 5d$.

Sometimes the water columns are inconveniently too long, then hydraulic pressure gauges should be set up and readings taken on a dial; but this arrangement is temporary and tedious. To avoid this difficulty, Mr. Metcalf invented a double gauge as shown in Fig. 59.

Fig. 59



h_5 = pressure head at centre of main pipe.

h_6 = pressure head at centre of Venturi throat.

It is clear $h_3 = h_5 - h_1$ and $h_4 = h_6 - h_2$

$\therefore \; h_5 - h_6 = (h_3 - h_4) + (h_1 - h_2)$.

The above are easily read on a scale of convenient length.

Venturi* meters are now manufactured in Europe with clock-work and dials which record the flow on section paper in terms of gallons per minute.

Sometimes gauges are put up as shown in fig. 59 and an attendant records the readings on a graduated scale, from which the discharges are known by reference to prepared charts.

Venturi Rate Controllers to regulate the discharge from modern Rapid Filter plants for water-supplies for cities in Europe are in use now.

84. *Transmission of power through pipes.*—If power is transmitted through a considerable distance by means of water under pressure, the power supplied will be in proportion to the quantity of water per second passing through the pipes and to the total head of water. As the water flows along the pipe, there is a loss of head due to friction.

The horse-power transmitted is a maximum when the head lost in friction is one-third of the total head supplied.

Efficiency of transmission $= \dfrac{H - h}{H}$, H being total head and h being head lost in friction.

Power is transmitted through water pipes for working hydraulic machines, but this has been now displaced by electricity, generally.

85. *Miscellaneous.* (*i*) *Corrosion of Pipes* :—All water mains, as they grow older, discharge less water under similar conditions, due to deposits on inner side of the pipe. Water in its natural stage contains iron and is acid in reaction. Salts held in suspension or in solution in water gradually deposit on the inner surface of the pipe and form incrustations and impede the flow of water.

Slime, a deposit of black substance, occurs not only in pipes but also in tunnels, brickwork channels. Slime contains iron and is of organic growth.

Sometimes the deposits are very readily brushed out of the pipes by means of rotary brushes.

* Venturi was an Italian Mathematician of very high order.

The pipes are painted with antirust paint such as asphaltic coating, or bitumen coating in smooth and perfect layers. Doctor Angus Smith's solution has been found to be a very good antirust paint on the market.

On the Tansa steel mains for Bombay Water Supply, Bitumastic paint (Bouranite) was used by hand and not by sprayers which affect the lungs of the workmen.

Percentage of decrease in discharge due to incrustation per year has been worked out by some engineers in America and by Mr. Bruce (Water Works Engineer to Bombay Municipality in the last quarter of nineteenth century), and their results are given as under. The pipes were coated with Dr. Angus Smith's solution (P. I. C. E., Vol. 162, p. 139).

Diameter of pipe in inches.	Age in years.	Percentage of decrease in discharge per year.
16	8·5	1·8
24	15	0·9
24	24	1·3
32	42	0·8
48	8	2·8
48	10	0·7

To allow for the diminution in discharge caused by corrosion, a pipe should be designed to give an initial discharge in excess of the requirements, by the following allowances, Q being the discharge required (Barnes).

Uncoated cast iron	1·55 Q	Asphalted cast iron	1·45 Q
Asphalted rivetted pipes	1·33 Q	Wood stave	1·08 Q
Neat cement or concrete	1·06 Q		

(*ii*) *Thickness of cast iron pipes* :—Pipes break either from earth pressure or water pressure or from water hammer. Minimum thickness is given by Unwin as under :—

$t = 0{\cdot}11 \sqrt{D} + 0{\cdot}10$ inches : D being diameter in inches.

It has been found that while small pipes of adequate thickness, have ample strength to resist ordinary internal pressures, they are relatively speaking, more likely to be fractured by straining actions, arising from unequal earth pressures than the larger sizes.

To stand different pressures due to different heads of water, pipes are now made in different classes of thickness and weights by makers, whose catalogues give complete information on the subject. The Government of India have complete specifications on this subject and also about the details of the joints of the pipes.

(*iii*) *Wrought iron pipes.*—These are used in hilly countries with screwed ends, fitted into screwed sockets for small diameters (4" to 8"), the advantage being that they adapt themselves to the surface of the ground and can be easily removed when required. They are also useful for boring purposes and tube-wells 2 inches to 12 inches diameter.

Lately Victaulic joint,* consisting of a piece of a pipe with double socket and a rubber ring inside has been invented. This allows the pipes to sag to suit ups and downs of ground surface without leaking.

For supplying water to house taps inside houses W. I. pipes ½" to 2" diameter, are used. Discharges through these are given in nomogram No. IV, Fig. 61B.

(*iv*) *Steel pipes.*—For high pressures, where cast iron pipes would be too thick and too heavy to handle and too expensive also, steel pipes are used. For waterworks in America steel pipes are frequently used.

Discharge through steel mains is calculated by the usual formula $V = C \sqrt{r} \sqrt{s}$; C, the coefficient varies from 78 to 120, depending upon the diameter of the pipe, the type of the joint, the thickness of the plates (because this affects the size of rivet head), the method of rivetting and the smoothness of the inside of the pipes ; see the last para of the Article No. 71 on this subject.

* The writer laid two miles of 4" W. I. pipe with victaulic joints to bring potable water from far off source for staff for salt mines at Warcha in West Punjab (1930).

For further useful information on steel pipes, see Journal of the Institution of Engineers (India), Vol. III, December 1927.

British Engineers use thicker pipes than the American Engineers whose thin pipes get deformed under earth pressure in trenches.

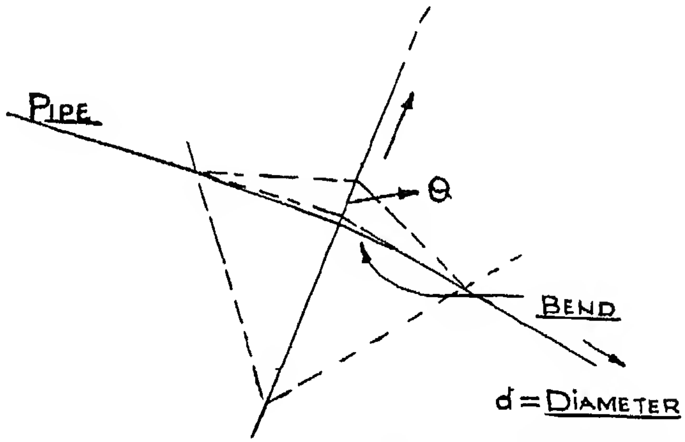
The writer, while employed as District Engineer in 1912/13 on Tata-Hydro-Electric Scheme at Khandala near Bombay, had chance to supervise the laying of 6 feet diameter steel main along a hillside about 6,000 feet long. The joints were flanged and rivetted. The steel plates are joined also by rivets; at the lower end the pips line bifurcated into 3 main lines, each of about 36″ diameter and about 6,000 feet long. The total head at the lower end where the three small main lines enter the power house at Khopoli for driving water turbines is 1,700 feet. The thickness of these steel pipes is calculated in different lengths according to the pressures they have to stand. The pipes were supplied by Messrs. Escher Wise & Co., of Switzerland whose engineers laid the pipes in position also.

The pipes are supported on masonry blocks and do not touch the earth surface in the trench or hill side which are well drained to prevent collection of water due to rain or otherwise.

The pipes are well painted with antirust paint, inside and outside. and exposed to air.

The pipes are also anchored. At each point where the pipe line, which follows the available slope of the ground, has a change in the grade, a masonry anchor is built on the top of the pipe. It is a simple block of masonry founded on good ground about 10 feet to 12 feet below the underside of the pipe and carried about 10 feet above the top of the pipe. The thickness of this block depends upon the hydraulic pressure inside the pipe, tending to lift it up. Holding-down steel rods are embedded in the block and bolted to saddles on pipe to keep it in position.

Fig. 60



Outward pressure, tending to throw the bend out as shown in fig. 60 is given by the formula,

$$P = 2\,\frac{\pi d^2}{4}\,62\cdot5\,H \sin\frac{\theta}{2}\ \text{lbs.} \qquad \ldots\ldots 69.$$

Where H = pressure at the bend. If the velocity is great, H should be increased by $\frac{V^2}{2g}$.

At Jogindarnagar in Mandi state, steel pipes are used to carry water from a great height to power-house to work turbines. The pipes are laid on blocks and exposed to air and not covered with earth. Expansion joints are also used, and they are painted with white shining paint.

At Kilokree sewage pumping station for New Delhi in India, 30″ steel main was laid by the writer in 1925 from pumps to a tank at a higher level; one expansion joint was laid at the start. The pipe was well painted with antirust paint both inside and outside. The joints were flanged and rivetted. To prolong the life of the pipes, it was surrounded by a six inches coat of fine cement concrete (1—2—4) all round. The pipes were laid in a trench 10 feet deep and finally covered with earth, the trench being filled upto natural ground surface.

Experience teaches that corrosion of pipes does not usually produce diminution in the thickness of the steel plates but occurs in patches and small pinholes before the plates are eaten up right through.

The thickness of the steel pipes is determined by the manufacturers, taking into consideration the pressure head the pipes have to stand, and several other factors. British Engineers adopt 3/8ths of an inch thick plates at least.

Herschell's experiments on asphalted rivetted steel pipes 3'-6" diameter, gave value of f in the formula 52A above, as ·00,675 with a velocity of 1 ft. per second to ·0055 with a velocity of 6 ft. per second. After 4 years use, f varies from ·0085 to ·0065 with the same range of pipes.

(v) *Wooden Stave Pipes :—*

Hard wood is used for making water mains, where available and cheap. Wooden pipes upto 72 inches in diameter have been made and used in America. The pipes are made of wooden staves machined to proper curvature and joined at ends by iron tongues. Hard and well seasoned timber must be used. The outside should be coated with some wood preservative and strengthened by steel bands, preferably oval in cross section, to take up the tensile strain. It is not necessary to coat these pipes on the inside with any paint. Care should be taken to see that the pipes are always full and never empty. There is very little incrustation on the inside compared to C. I. pipes.

The pipes rest on wooden shoes. The pressure head, which pipes are required to stand, should be about 120 feet and not more than 200 feet, relief arrangements being made for water hammer.

Timber for pipes should be straight, long in grain and fairly flexible, specially in curved portions of the pipe.

At Srinagar in Kashmir State, India, the writer saw wooden stave pipes of about 18" diameter in use for a part of the long gravitating main for the water supply of that city, which is 200 miles away from the nearest railway station Jammu-Tavi and where plenty of timber is available.

(vi) *Power Pipe Lines :—*From Khandala Hotel to Khopoli Power House (Tata Hydro-Electric Scheme at Lonavla near Bombay, gross output 100,000 H.P.) : 6 ft. diameter steel power pipe line was laid by the writer in 1912/1913 for a length of about 8,000 feet, and then split up in three lines of about 36" diameter for a length of about 4,000 ft. to work the turbines : Total fall about 1,700 feet. Supply of water from the Storage Tanks about 500 Cusecs.

The diameters of these steel single rivetted pipes were so fixed as to lead to the minimum overall cost of operation and maintenance. The interest on capital cost, maintenance, depreciation together with the value of the power lost due to friction in pipes were also considered. Theoretical Horse-Power $= \dfrac{500 \times 60 \times 1700 \times 62{\cdot}5}{33{,}000} = 97{,}000$ H.P. Actual Nett output from the generators in 1948 = 60,000 K. Watts. One K. W. for one hour is one commercial unit. $60{,}000 \times 24 =$ 1,440,000 units in 24 hours. This is the output in Power House. Losses in transmission also count, as the electrical energy is carried to Bombay on transmission lines.

(vii) *Pipes carrying water charged with sand :—*Experiments on Sand, having grains varying from ·16 m.m. to ·75 m.m. in diameter show that for a 1-inch pipe, the sand carrying velocity is about 3·5 feet per second while for a 3-inch pipe it is about 4 feet per second and for a 32-inch pipe about 9 feet per second. With fine sand, the additional loss of head is 25 per cent over and above the loss due to water alone for each 1 per cent of added sand. The proportion of sand to water may exceed 50 per cent if the velocity is more than necessary to cause suspension.

(viii) *Diaphragm in a Pipe Line :—*Let D be the diameter of the pipe line with flanged joints. Insert a diaphragm of a thin metal plate with a circular or rectangular hole of diameter d. Take the pressure with a hydraulic gauge both on upstream and downstream sides of the diaphragm. Let these pressures be h_1 and h_2. Let the ratio of $\frac{D}{d}$ be n : The discharge through the orifice in the diaphragm is

$$Q = \frac{C \pi D^2}{4} \left(\frac{2 g h_1 - h_2}{n^2 - 1} \right)^{\frac{1}{2}} ; \; C = {\cdot}614 \text{ to } {\cdot}634.$$

86. *Examples :—*

(1) Find the approximate discharge in gallons through a pipe 4 feet in diameter having a fall of 1 foot per mile.

In this case the hydraulic slope is the same as the actual slope of the pipe line, because the water enters at atmospheric pressure and is discharged at atmospheric pressure.

See formula 49 : $V = 39\sqrt{ds}$: $d = 4$ ft. : $s = \frac{1}{5280}$: $Q = A \times V = \frac{\pi d^2}{4} \times 39\sqrt{ds} = 5017$ G/minute.

If the pipes are incrusted Nomogram No. 3, Fig. 61 A gives discharge 5 ,500 gallons per minute, by Unwin's formula.

(2) A pipe 4,800 feet long is to be laid with a fall of 1-192 on a sloping ground and is required to deliver 3,250 gallons per minute at a pressure of 10 lbs per sq. inch at the lower end. The head over the centre of the inlet orifice is 10 feet. Find out the diameter of the pipe.

Total Fall in 4,800 feet is $\frac{4800}{192} = 25$ feet. Total available head $= 25 + 10 = 35$ feet.

10 lbs. per sq. inch pressure head is required at the point of delivery. This means $\frac{10 \times 144}{62 \cdot 5} = 20$ feet approximately.

Net available head is $35 - 20 = 15$ feet, to meet frictional losses.

Thus the hydraulic grade is $\frac{15}{4800}$ or 1 in 320.

By formula 52B.

$$d = \cdot 2545\sqrt{\frac{Q^2}{S}} = 2 \text{ feet, } Q \text{ being } 8 \cdot 66 \text{ cusecs. and } S = \frac{1}{320}.$$

Nomogram No. 3 Fig. 61A also gives 24 inches the required diameter of the pipe.

(3) Find the discharge of a wrought iron pipe, whose diameter is 3 feet and slope 1 in 1,000.

See article 70 : Table for values of C, gives C as equal to 125 in this case.

$$V = C\sqrt{r}\sqrt{s} = 125\sqrt{0 \cdot 75}\sqrt{\frac{1}{1000}} = 3 \cdot 35 \text{ ft./second}$$

$Q = \text{Area} \times \text{Velocity} = 7\ 067 \times 3 \cdot 35 = 23 \cdot 67$ cusecs.

(4) An open channel discharging 16 cubic feet per second is passed under road through a syphon of smooth plastered brickwork of section 2 ft. × 2 ft., which first descends 10 feet vertically then travels 80 feet horizontally, again rises 10 feet vertically, the bends being right-angled and sharp. What is the loss of head in the tunnel ?

Area of water way $= 2 \times 2 = 4$ sq. feet.

Discharge $Q = 16$ cusecs Therefore $V = \frac{16}{4} = 4$ ft. second

There are 4 elbows of 90 degrees each, see article 73 : Loss of head per each elbow $= \frac{1}{2}\ \text{Sin}^2\theta\ \frac{V^2}{2g} = \frac{1}{8}$

$= 0 \cdot 5$ feet for 4 elbows. .... *(a)*

The length of the syphon barrel is 80 feet.

This is a case of a short pipe. Apply formula 52A.

head lost, $h = \mu\frac{4LV^2}{2gd} = 0 \cdot 19$ feet .... *(b)*

$\mu = \cdot 005\left(1 + \frac{1}{12d}\right) = \cdot 0052$: $d = 2 \cdot 25$ ft. . $L = 80$ ft. : $V = 4$ ft. per second : $g = 32$ feet.

The head absorbed in overcoming resistance at entrance and to generate velocity V at start is $1 \cdot 5\ \frac{V^2}{2g} = 0 \cdot 4$ .... *(c)*

Thus the total loss of head

$= (a) + (b) + (c) = 0 \cdot 50 + 0 \cdot 19 + 0 \cdot 40 = 1 \cdot 09$ feet.

(5) A 4″ branched main in a street supplies water to each of the houses in the street through $\frac{3}{4}$ inch service pipe, one for each house. The highest point at which the service pipe delivers water is 34 feet above the water main. The service pipe is 72 feet long If the pressure in the main is $15\frac{1}{2}$ lbs. per square inch, how many gallons per minute will the service pipe deliver at its top end ? How many houses would the main supply ?

Pressure head in the main $h = \frac{P}{W} = \frac{15\frac{1}{2} \times 144}{62 \cdot 5} = 36$ feet.

Hydraulic gradient in the service pipe is equal to $\frac{36-34}{72} = \frac{1}{36}$ or 1 in 36.

Diameter of service pipe $d = \frac{3}{4}$ inch $= \frac{1}{16}$ feet.

$$\mu = \cdot 01 \left(1 + \frac{1}{12d}\right) = \cdot 023 : \ C = \left(\frac{2g}{\mu}\right)^{\frac{1}{2}} = 53 : \text{article 70.}$$

$$V = C\sqrt{r}\sqrt{s} : Q = A \times V = \frac{\pi d^2}{4} C \sqrt{\frac{d}{4}} \sqrt{s} : S = \frac{1}{32}$$

$= \cdot 00338$ cusecs or $1\frac{1}{4}$ gallons per minute.

Let n be the number of houses that can be supplied with water from the 4 inch main. By equation 58, the discharge Q varies as $d^{\frac{5}{2}} s^{\frac{1}{2}}$: as the total discharge through all the service pipes is equal to the total discharge through the 4 inch branch and $s^{\frac{1}{2}}$ is the same for all services,

$\therefore (n \times \frac{3}{4})^{\frac{5}{2}} = (4)^{\frac{5}{2}}$ or $n = 65$.

(6) Find out the discharge of a syphon $3\frac{1}{2}$ feet in diameter, 240 feet long, the difference of water level being 12 feet.

H, the total loss of head (12 feet) consists of 3 parts; *a*, loss of head at entry : *b*, loss of head at two 90° bends : *c*, loss of head through the pipe 240 feet long.

Suppose (*a*) and (*b*) do not exist at all.

Then by equation 54.

$$V = \sqrt{\left(\frac{2gHd}{1 \cdot 5d + 4\mu L}\right)} \quad : \quad d = 3 \text{ ft. } 6 \text{ inches} : H = 12 \text{ feet} : \text{and } L = 240 \text{ ft.} :$$

$$\mu = 0 \cdot 1\left(1 + \frac{1}{12d}\right) = \cdot 0102 : \ V = 16 \text{ feet.}$$

This is so if (*a*) and (*b*) do not exist but they do exist: hence V must be less than 16 feet per second : assume V = 10 feet per second. In this case

$$(a) = 1 \cdot 5 \frac{V^2}{2g} = 2 \cdot 34 \text{ feet.}$$

$$(b) = (\tfrac{1}{2} \text{ sine } \theta^2) \frac{V^2}{2g}, \quad \theta \text{ being } 90° \text{ in this case (article 73).}$$

$$= \tfrac{1}{2} \times \frac{100}{64} = \frac{50}{64} \ \therefore \text{ for 2 bends} = \frac{50}{32} = 1 \cdot 56 \text{ feet.}$$

(C) see equation 52A.

$$h = \mu \frac{L}{r} \frac{V^2}{2g} : \mu = \cdot 0102 : V = 10 : L = 240 \text{ ft. } r = \cdot 875$$

$= 4 \cdot 40$ feet.

$(a) + (b) + (c) = 2 \cdot 34 + 1 \cdot 56 + 4 \cdot 40 = 8 \cdot 30.$

Thus with V as 10 feet per second, the total loss of head $= 8 \cdot 30$ feet against actual loss of 12 feet.

By equation 53, $\frac{\text{Actual velocity}}{\text{Assumed velocity}} = \left(\frac{\text{Actual head}}{\text{estimated head}}\right)^{\frac{1}{2}}$

$$\therefore \text{actual velocity} = 10\sqrt{\left(\frac{12}{8 \cdot 30}\right)} = 10(1 \cdot 44)^{\frac{1}{2}}$$

$= 12$ feet per second.

$$\text{Discharge} = Av = \frac{\pi (3\frac{1}{2})^2}{4} \times 12 = 115 \cdot 45 \text{ cusecs.}$$

Example 7.—A pipe 2 feet in diameter has a fall of 1 in 1000 for half a mile, after which it falls at the rate of 1—250 for quarter of a mile. If the level of the water in the supply cistern stands at 5·13 feet above the centre of the pipe at the upper end, what will be the discharge per minute?

Fig. 61

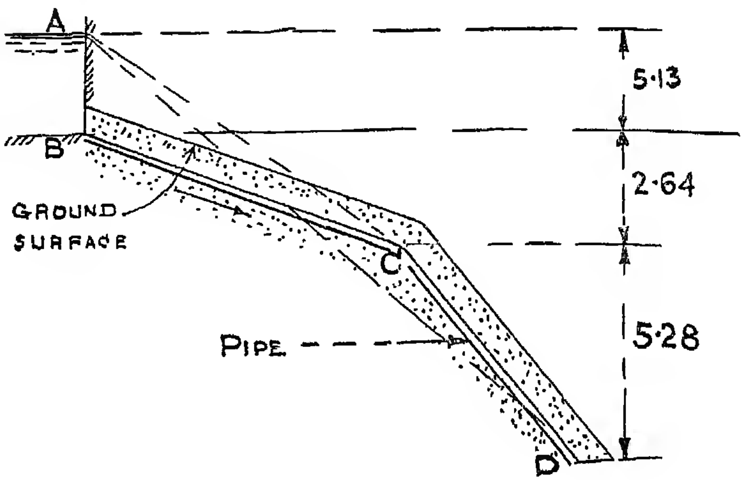


Fig. 61 shows the arrangement of pipes.

The pressure heads at the points B, C, D, are 5·13, 7·77, 13·05 feet respectively. The hydraulic gradient for the pipe line BC, is line AC; because the point C is above the mean H. G. line AD. Also therefore the line CD is the H. G. line for pipe line CD. Thus the H. G. line AC regulates the discharge through the whole pipe line B C D.

$$Q = \frac{\pi d^2}{4} \times 39\sqrt{ds} \text{ : see equation No. 40 : } s = \frac{7\cdot77}{2640}$$

$= 9\cdot4$ cusecs.

Ths segment C D, of 24″ diameter pipe will not run full. Its slope, $\frac{5\cdot28}{1320}$ is hydraulic grade. Design the diameter of this pipe to carry 9·4 cusecs with the available hydraulio grade which means its actual slope. In article 74 it is explained that 's' varies as $\frac{1}{d^5}$, for the same discharge.

For the pipe line BC, d, the diameter is 2 feet. $s = \frac{7\cdot77}{2640}$. For the pipe line CD, diameter d_1 is to be found. Its slope $S = \frac{5\cdot28}{1320}$ or $\frac{10\cdot56}{2620}$

$$\therefore \frac{d_1^5}{d^5} = \frac{7\cdot77}{10\cdot56}$$

$$\text{or } \frac{d_1}{2} = \left(\frac{7\cdot77}{10\cdot56}\right)^{\frac{1}{5}} \quad \therefore d_1 = 1\cdot88 \text{ ft. say 22 inches.}$$

The problem can be easily solved from the Nomogram No. 3. Fig. 61A :

Example 8.—Two reservoirs are connected by a straight pipe line, one mile long. It consists of two sections, each half a mile, but of different diameters, say 6 inches to three inches rsspectively; there being a sudden change from 6 inches to 3 inches. The surface of the water in the upper reservoir is 100 feet above that in the lower. Find cut in detail the losses of head that occur at points of entry, change and dischargs; also the different sections. Find the discharge in cusecs ?

Nomogram No. 3 will not help here.

Since discharge is constant right through, the velocities V_1 and V_2 in 2 sections are in the inverse ratio of the squares of the diameters.

$$i.e. \frac{V_1}{V_2} = \frac{(d_2)^2}{(d_1)^2} = \tfrac{1}{4} \therefore V_1 = \frac{V_2}{4}$$

$$\text{Head lost at entrance} = \frac{0\cdot5V_1^2}{2g} = \cdot031\frac{(V_2)^2}{2g} \quad \ldots\ldots(a)$$

Head lost in 6″ pipe due to friction (Sse equation 52A).

NOMOGRAM No 3. FIG 61A

OF

DISCHARGE OF INCRUSTED C.I. PIPES

BY UNWINS FORMULA : $S = \frac{0.0440}{D^{1.16}} \frac{V^2}{2g}$

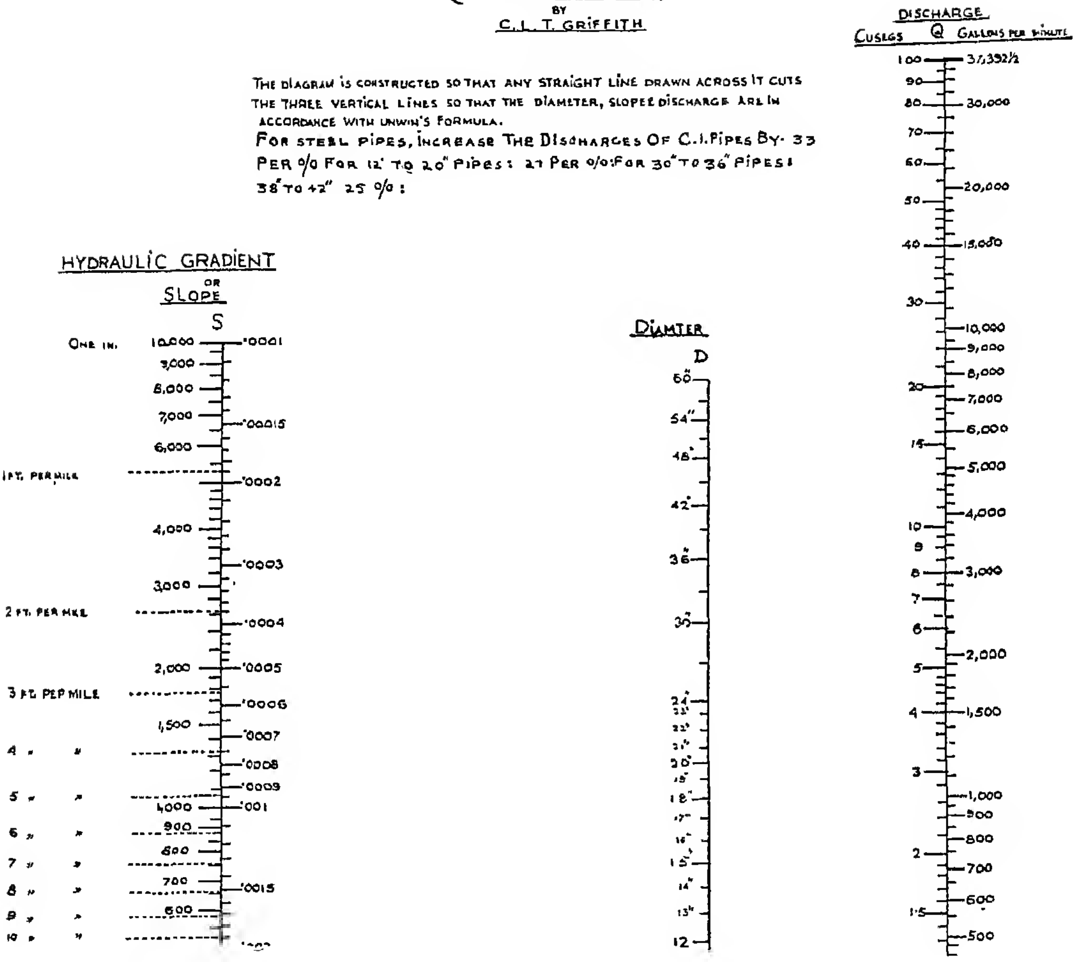
(g TAKEN AS 32.125 FOR INDIA)

BY

C. L. T. GRIFFITH

THE DIAGRAM IS CONSTRUCTED SO THAT ANY STRAIGHT LINE DRAWN ACROSS IT CUTS THE THREE VERTICAL LINES SO THAT THE DIAMETER, SLOPE & DISCHARGE ARE IN ACCORDANCE WITH UNWIN'S FORMULA.

FOR STEEL PIPES, INCREASE THE DISCHARGES OF C.I. PIPES BY 33 PER % FOR 12" TO 20" PIPES: 27 PER %: FOR 30" TO 36" PIPES: 38" TO 42" 25 %:



$$= \mu \frac{L}{r} \frac{V_1^2}{2g} = \frac{\cdot 01 \times 2640 \times V_2^2}{16 \times 0\cdot 125 \times 2g} = \frac{13\cdot 2\, V_2^2}{2g} \quad \dots\dots (b)$$

r being Hydraulic mean radius, μ being ·01.

Head lost at sudden contraction $\frac{0\cdot 5\,(V_2)^2}{2g}$ ......(c)

Head lost in 3″ pipe due to friction $= \mu \frac{1 \times (V_2)^2}{r \times 2g}$

$$= \frac{\cdot 01 \times 2640 \times (V_2)^2}{\cdot 0625 \times 2g}$$

$$= \frac{422\,(V_2)^2}{2g} \quad \dots\dots (d)$$

Head lost at exit $= \frac{(V_2)^2}{2g}$ ......(e)

Total head lost 100 feet = a + b + c + d + e

$$= \frac{(V_2)^2}{2g} (\cdot 031 + 13\cdot 2 + \cdot 5 + 422 + 1)$$

$$= 436\cdot 73 \frac{(V_2)^2}{2g}$$

$\therefore \quad V_2 = 3\cdot 28$ feet per second.

Discharge $\quad Q$ = area of 3″ pipe $\times V_2$

$= \cdot 049 \times 3\cdot 28 = 0\cdot 1607$ cusecs.

Example 9.—Two cylindrical tanks A (5 feet diameter) and B (10 ft. diameter) are connected by a short pipe 4 inches diameter with bell mouth inlet. At the beginning the level in A is 10 feet and in B is one foot above the centre line of the pipe. In what time will the surface levels be the same ?

Answer 138 seconds.

Example 10.—Ascertain the thickness of C. I. Pipe 24″ internal diameter, subject to internal water pressure of 100 feet head : Length to be assumed 1 foot. Working Tensile strength of cast iron f = 3500 lbs. per square inch.

Consider a plane passing horizontally through the central axis of the pipe, dividing it into two semi-cylinders, one being stressed upwards and the other being stressed downwards, by equal and opposing forces, each being $100 \times 62\cdot 5 \times 2 = 12{,}500$ lbs., working on a length of 1 foot: If t be the thickness of the pipe in inches, the resisting force, opposing the rupturing stress of 12,500 lbs. at the intersection of the Horiz-plane with the pipe, should be 2t × 12 inches × 3500.

$$t = \frac{12500}{2 \times 12 \times 3500} = 0\cdot 15 \text{ inches.}$$

f = 2000 lbs. per sq. inch to 3000 lbs. for 2″ to 8″ C. I. Pipe : 3500 lbs. for 24″ pipe : for wrought iron f = 7500 to 8500 lbs. for steel f = 10,000 to 12,000 lbs. In the case of rivetted pipes, the efficiency of the rivetted joint = 55 per cent for single and 70 per cent for double rivetting.

Example 11.—A single riveued steel pipe, 30 inches internal diameter, is exposed to a head of 450 feet of water. Taking f = 12,000 lbs. per square inch, and the efficiency of the joints = 60 per cent, what should be its thickness ?

Ans. $t = \cdot 407$ inch $= \frac{13}{32}$ inch.

Example 12.—A cast-iron pressure pipe, 4 inches diameter, is exposed to a pressure of 1,100 lbs. per square inch. Taking f = 2,800 lbs. per square inch, what is the requisite thickness ?

Ans. $t = 1\cdot 032$ inch $= 1\frac{1}{32}$ inch.

Example 13.—In a 12 inch Venturi meter the throat diameter is 4 inches. Taking C to be constant = ·982, determine the difference of head at the entrance and throat when discharging.

$$\left.\begin{array}{ll} (a) & 100 \\ (b) & 500 \\ (c) & 1000 \end{array}\right\} \text{gallons per minute.}$$

N. B.—1 gallon = 10 lbs.

Answer : (a) Vel. = ·340 f.s. .. ·149 feet
(b) „ = 1·701 f.s. .. 3·73 feet
(c) „ = 3·402 f.s. .. 14·92 feet.

CHAPTER XI.

OPEN CHANNELS. UNIFORM FLOW. MORE OR LESS CLEAR WATER.

87. Preliminary.

Fig. 62

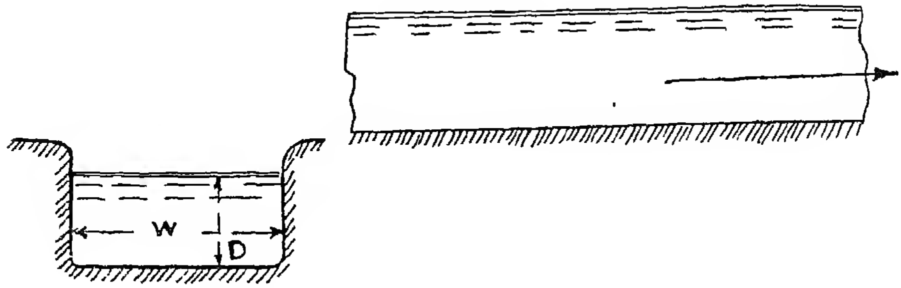


Fig. No. 62 shows an open channel with a uniform bed slope, uniform cross-section and uniform depth of flow. The surface water is subjected to Atmospheric pressure all along.

The flow of water in the channel takes place due to force of gravity acting on the volume of water in the direction of flow. This force of gravity is spent in overcoming the friction offered by the bed and sides of the channel to the flow of water ; also in keeping the mass of water in motion. The force of gravity increases as the bed slope gets steeper.

The wetted bed and sides of the channel, form the wetted perimeter, denoted by B, $= w + 2D$.

Area of the channel divided by the wetted perimeter gives the ratio, called Hydraulic mean radius, denoted by $r = \frac{A}{B} = \frac{DW}{W + 2D}$, the sides of the channel being supposed to be vertical, W and D being, width and depth.

The ratio $\frac{r}{D} = \frac{W}{B}$ and $A = r\,B$. If the depth of flow D increases, A increases faster than W and r therefore increases ; but $\frac{W}{B}$ decreases so that r increases less rapidly than D.

For small changes of water level, r and D both change at about the same rate.

In the above case the slope of the water surface is parallel to the bed slope of the channel and is called virtual slope.

A pipe running not full is classed as an open channel.

88. *Velocity.*—The rate at which the water moves down the channel is called velocity of flow, denoted by the letter V and measured in so many feet per second. The velocity is different at different points of the water surface and at different depths below the water surface in the same cross-section.

Chezy has given the following formula to find out the mean velocity of flow in a uniform channel.

$V = c\,\sqrt{r}\;\sqrt{s}$ 70

r = Hydraulic mean radius in feet.

s = Sine of slope of Channel.

V = feet per second.

C = coefficient depending upon the nature of the inside surface of the channel.

It is not proposed to go into the mathematical investigation of the formula. It is accepted nowadays all over.

Kutter and Ganguillet, two Swiss Engineers, investigated the matter thoroughly and have given the value of C as under, in the formula $V = c\sqrt{r}\ \sqrt{s}.$

$$C = \frac{41\cdot6 + \frac{\cdot00281}{s} + \frac{1\cdot811}{n}}{1 + \frac{n}{\sqrt{r}}\left(41\cdot6 + \frac{\cdot00281}{s}\right)} \text{ in foot units.}$$

s = Bed slope of Channel.

r = Hydraulic mean radius in ft.
n = coefficient of rugosity, called Kutter's n

(Kutter and Ganguillet two swiss Engineers, made exhaustive experiments on the flow of water in Mississippi river, for good many years.)

The values of n are given on Table No. I.

It is said that Kutter did not originally propose to apply his formula to pipes but Engineers make use of this formula as a matter of convenience. For discharge through channels, Kutter's formula is freely used by Engineers in the British Empire. Major Cunningham, R.E., in his observations on Ganges canal in 1888 conclusively proved that Kutter's formula is very reliable.

The formula given by Manning is probably as good as that of Kutter, the value of n being the same as that of Kutter.

Manning's formula is $V = \frac{1\cdot486\ r^{0\cdot67}\ s^{0\cdot5}}{n}$

n has the same value as in Kutter's.

Bazin, a French mathematician, after long and exhaustive experiments gave in 1897 the values of C in the Chezy formula $V = c\sqrt{r}\ \sqrt{s}$ as under :—

$$C = \frac{157\cdot5}{1 + \frac{K}{\sqrt{r}}} \text{ foot units} \quad \ldots\ldots 71.$$

Where K is a constant depending upon the surface of the channel and has following values

	K
Smooth cement plaster	·109
Clean smooth sides of wood, brick, stone	= 0·29
Dirty sides of do. do. do.	0·50
Sides of natural earth (ordinary rough earth canals)	2·35
Rubble masonry	0·833
Earth Channels of very good surface	1·54
Very rough channels of earth with weeds on sides	3·17

The above formula gives as good results as that of Kutter but is not in favour of British Engineers in British Empire, though French and German Engineers use it freely. It is given here in memory of the Great Mathematician M. Bazin.

TABLE No. I.

VALUES OF 'N' IN KUTTER'S FORMULA.

Specification of the Channel	Value of 'N'.	Authority.
Timber well planed and perfectly continuous	0·009	
Planed timber, not perfectly true	0·010	
Glazed and enamelled materials with no irregularities, or clean coated pipes	0·010	
Pure cement plaster	0·010	
Wood-stave pipes	0·010	
Plaster in cement, one-third sand	0·010	
Pipes of iron, cement or terra-cotta, well jointed, and in best order ..	0·011	Kutter
Timber unplaned and continuous, new brick-work : ordinary iron pipes	0·012	
Good brick-work, and ashlar, unglazed stone-ware, and earthenware ..	0·013	Kutter
Canvas lining on wooden frames	0·015	Kutter
Foul and slightly tuberculated iron, rough faced brickwork	0·015	
Well dressed stonework	0·015	
Wooden troughs with battens inside, ½ inch apart	0·015	
Fine gravel, well rammed	0·017	Kutter
Rubble masonry in cement, in good order	0·017	
Tuberculated iron pipes, brickwork or stonework in inferior condition ..	0·017	
Earthen channels in faultless condition	0·017	
Ditto, during heavy silting	0·017	
Earthen channels in very good order	0·018	
Coarse gravel, well rammed	0·020	Kutter
Large earthen channels maintained with care. Channels in earth in good regime	0·0225	Punjab Irrigation Branch
Small earthen channels maintained with care. Channels in earth in good regime	0·025	Punjab Irrigation Branch
Channels in average order	0·025	Kutter
Channels in order, below the average	0·0275	Jackson
Channels in bad order	0·030	Kutter
Channels in very bad order	0·035	Kutter
Channels of worst possible character with turbulent flow and large obstructions	0·040	Jackson
Torrents encumbered with detritus and boulders like those in Kangra Valley, Punjab	0·050	

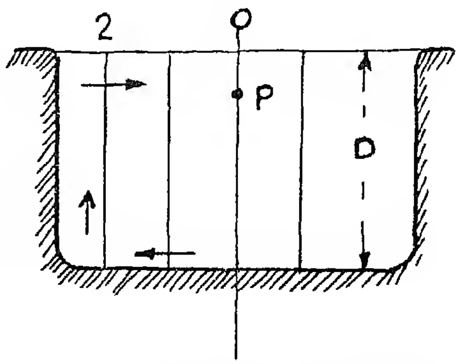
TABLE No. II.

C = Kutter's Coefficients for different slopes (Bellasis page 173).

Value of r	Kutter's Coefficients for different slopes.			
	N = ·010		N = ·030	
	Slope 1 in. 10,000	Slope 1 in. 1,000 and steeper slopes.	Slope 1 in. 10,000	Slope 1 in. 1,000 and steeper slopes.
0·5	126	138	33	36
1·0	148	156	42	45
2·0	168	172	52	54
4·0	186	185	64	63
6·0	195	191	70	68
10·0	206	197	78	74

89. *Relative Velocities in cross-section.*—Investigations have been made from time to time to find out the relation existing between velocities of different particles of water in a cross-section of the stream of flowing water. Results, approximately correct are given below.

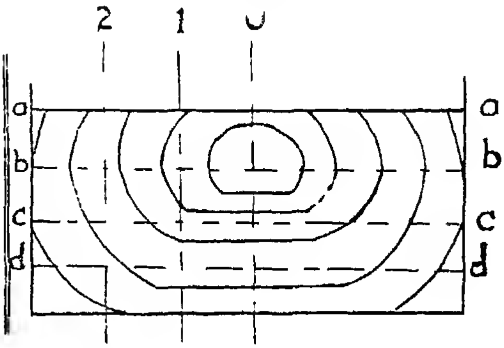
Fig. 63



Eddies rise from the bed to the surface. The water, of which the eddies are composed, is slow moving, and though the eddies retard the velocities at all points which they traverse, they have most effect at the surface because they spread out and accumulate there, specially at the central portions of the stream.

Fig. 63 shows that near the side, there is an upward current and there is a surface current from the side outwards, which causes the floating matter to accumulate in midstream. The currents are the result of eddies. I personally observed this phenomena in a canal for several days

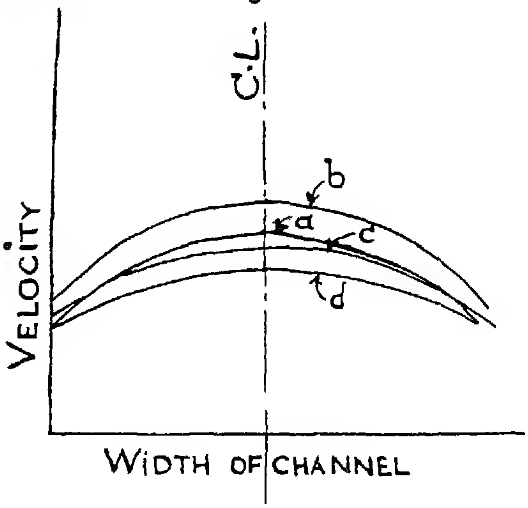
Fig. 64



It is eddies and cross currents that affect the velocities at different points in a cross section.

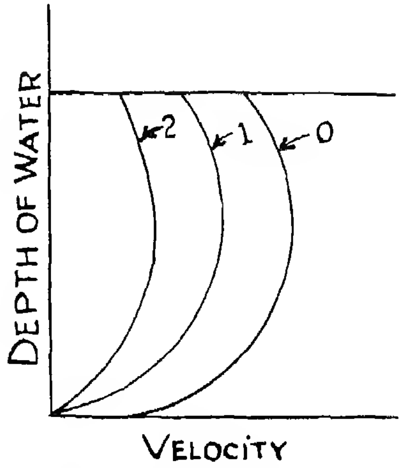
Figs. 64, 65, 66 show the velocity curves in cross-section of the stream. These are known in a general way, but not with accuracy, because their equations are not known.

Fig. 65



In fig. 63, the maximum velocity on the surface is at the point 0, the middle point of the section. This decreases as we go from the point 0 towards the sides due to resistance offered by the sides to the flow of water.

Fig. 66



The frictional resistance between the water surface and the atmosphere causes a slight reduction of velocity at the surface. The maximum velocity in the cross section of the channel will occur at the point P, a little below the surface (Fig. 63).

The velocity is least in the neighbourhood of the bed and banks and greatest in the axis of the stream at a point P, 0·3 D below the surface. See fig. 63 ; as the point P in the same cross section moves towards sides, its position becomes 0·5 to 0·6 D below the surface.

If Vs be the greatest surface velocity, Vb = the bottom velocity and V be the mean velocity, approximately their relation is $V = \cdot 8\ V_s = 1 \cdot 3\ V_b$ ....72.

The relation between V and Vb enables us in designing a channel, to assign a velocity which will not be injurious to bed and banks of known consistency of soil. The following mean velocities in feet per second are generally followed.

Clay	0·75	Boulders	4·00
Sand	1·50	Stratified rock	6·00
Pebbles	3·00	Hard rock	10·00

On the above subject exhaustive experiments were made by Major Cunnigham on Ganges canal near Roorkee and results can be obtained from the U. P. Irrigation Department.

The variation of velocities over the cross section of a channel, nearly rectangular, was investigated by Bazim in a channel 6 feet wide and 1·5 feet deep. This is shown in fig. 64. The curves are lines of equal velocity ; they have the greatest value at the centre, just below the water surface and decrease towards the sides and base.

Fig. 65 shows horizontal velocity curves in horizontal planes through points a, b, c, d, the velocities at different points of the section lines a, b, c and d are plotted on a base representing the width of the channel and the velocities as ordinates, the channel being supposed to run full. The shape of the curve depends upon the shape of the cross section and not on its sides. For further mathematical treatment of the subject, please see Hydraulics by Bellasis and Major Cunnigham's experiments on Ganges canal.

Fig. 66 shows the variation of velocities in vertical planes through vertical lines 0, 1 and 2. The horizontal ordinate represents the velocity and the vertical ordinate represents the depth. The *mean velocity on any vertical section occurs* at nearly 0·6 D, from the bed. It varies with the type of the channel and with the nature of the sides.

For the detailed mathematical treatment of the subject see page 166 of Bellasis Hydraulics.

If the surface velocities are observed by means of a float, the mean velocities are given by the following table. (Bellasis)

Values of ratio of mean velocities to observed velocity.

Hydraulic mean radius in feet.	Fine plaster Channel.	Cut stone or brick work.	Rubble or Boulder masonry.	Earth
0·5	·84	·81	·74	·58
to	to	to	to	to
5·0	·85	·83	·71 .	·76

If Vc be mean velocity on the central vertical and Vm that in the whole section,

then $\frac{Vm}{Vc}$ = specific ratio :

Ratio of mean width to depth of channel..	1	1·5	3	5	7	20	50	90
Specific ratio	·86	·87	·89	·91	·93	·95	·96	·98

Ratio of Mean to surface velocities on verticals away from the sides of the channel.

Depth on vertical ranging from ·9 to 28 feet. Value of n ranging from ·030 to ·010.

The above ratio varies from ·83 to ·92 (Bellasis).

Let V_s = maximum surface velocity feet per sec. V_m = mean velocity : Vb = bottom velocity : R = Hydraulic mean radius : Vm = K Vs (Bazin).

If R varies from 0·5 to 20, K varies from 0·84 to 0·85 for fine plastered surfaces.

from 0·81 to 0·84 for cut stone Brickwork.

from 0·58 to 0·80 for Rubble masonry, rough surface.

$Vm = Vb + 10{\cdot}87 \sqrt{R}\ \sqrt{s}$: s being bed slope (Morin).

90. *Velocity of entry.*—The entrance to a channel may be open to the source of supply or it may be closed by a head sluice. In the former case, there is, for a short distance, a rapid surface slope sufficient to generate velocity which the H. M. RADIUS and the fall maintain in the channel lower down ; in the latter the head is the difference of level between upper and lower sides of the sluice.

At the open inlet $V = C\sqrt{2gh}$; take C = 0·8.

$\therefore$ h (head to produce V at entrance) $= 1{\cdot}5\ \frac{V^2}{2g}$.

For a closed inlet let A be the area of sluice openings ; V = velocity through the sluices.

Since $V A = V_1 A_1$: Then $V_1 = \frac{A\,v}{A_1} = 1{\cdot}5\left(\frac{A}{A_1}\right)^2\ \frac{v2}{2g}$.

In both cases, the fall can be distributed if desired, by widening the channel near the entrance and thus reducing the velocity to be at first reduced.

91. *Bends.*—Bends in artificial channel are generally curves of large radius. For approximate loss of head in bends, Humphrey and Abott's Mississippi formula is adopted. If the arc of the bend subtends an angle of $\theta°$, $h = \frac{\theta}{90} \times \frac{\cdot 36\ V2}{2g}$ ....(73)

See paper No. 242 P. E. Congress (1941) by Mr. Wilson which is very interesting on this subject.

92. *Critical velocity.*—Critical velocity in a channel is that mean velocity which for a channel of given depth, will just keep the channel, all the year round, free from either silting or scouring its bed, when the water is running fully charged with silt upto the standard usually found in rivers.

The following table gives the critical velocities for varying depths for certain Punjab canals. In the case of channels in Sindh, drawn from Indus, the critical velocity has been assumed to be three-fourths of that applicable to canals in the Punjab.

TABLE III.

Critical Velocities for varying depths.

Depth in feet	Critical velocity	Depth in feet	Critical velocity.
1	0·84	9	3·43
2	1·30	10	3·67
3	1·70	12	4·12
4	2·04	15	4·75
5	2·35	20	5·71
6	2·64	50	10·27
7	2·92	100	16·00
8	3·18	..	..

As a canal in ordinary soil can not usually be made with greater velocity than about 3·5 feet per second, and as the critical velocity is greater than that in canals with a depth of over 9 feet, it is not desirable to design canals in ordinary soils which are neither to scour nor silt, with a greater depth than about 9 feet, unless the water is not silt laden or unless the silt it carries is such that the critical velocity is less than that given in the table.

93. *Channel Sections.*

(*i*) *Section of best form.*—A stream is of best form when for a given sectional area, the border is a minimum and the hydraulic mean radius therefore a maximum; velocity and discharge are greater than in any other stream of the same sectional area, slope, and roughness. The form which complies with this condition is a semicircle whose diameter coincides with the line of water surface.

The H. M. Radius is $= \frac{D}{2}$, D being the depth of flow. This form is used for concrete channels, or channels cut in rock. For circular channels, see Chapter XV on sewers and drains.

The best form for sloping sides is a semi hexagon, and still better the best form is the figure in which the bed and sides are tangents to a semicircle (described on the water line), specially when the side slopes are assigned.

Fig. 67

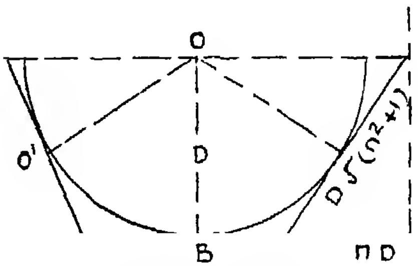


Fig. 67 shows such a channel.

B = Breadth of the base

D = Depth of water

$\frac{1}{n}$ = Slope of the sides

Length of sloping side $= D\ (n^2 + 1)^{\frac{1}{2}}$

$O\ O^1 = D$: The above channel gives maximum discharge for a given slope.

The mean Hydraulic radius $r = \frac{D}{2}$.

The above form is used for small earthen channels; with large channels, the depth becomes too great and impracticable.

The masonry duct in the sloping sides of hills at Khandala near Poona for Tata Hydro-electric scheme (output 100,000 H.P.) is of a trapezoidal *section; depth of water 9 feet: side slopes 1 in 4:* bed width about 14 feet: longitudinal slope of bed 1 in 2,000. Its length is about 3 miles. Discharge about 500 cusecs.

The side slopes of the form depend upon the nature of the soil through which such channels are cut.

(*ii*) *Rectangular channels.*—A Rectangular channel is a trapezoid with side slopes of 90°. The figure for maximum discharge will therefore be a half square. The H. M. radius is $\frac{D}{2}$, D being depth of the flow and half of the breadth. This form is employed for aqueducts of timber or masonry.

The masonry duct built by the writer in 1910 for Simla Hydro-electric scheme at Chhaba in the sloping sides of hills is a semi square, bed width 4 feet: depth of flow 2 feet: bed slope 1 in 1,000: velocity 3 ft./second: Dis. = 25 cusecs.

(*iii*). *Closed channels.*—This form of channel has boundaries on all sides of water section, the circle furnishes the best form.

Since this is the figure which has the least wetted perimeter for a given area, the Hydraulic mean radius or depth $= \frac{D}{4}$, where D is the greatest depth of flow. This form is generally adopted for pipe sewers. See article 113.

(*iv*). *Channels of constant velocity or discharge.*—If the channel is to maintain constant velocity when the depth of flow varies, it means that 'r' the hydraulic mean depth should be constant. This is practically impossible specially when the depth of flow decreases to zero. Even within the limits in which r is constant the mean velocity is not constant.

The velocity, as the depth of flow increases, is nearly constant in a very deep narrow channel with vertical sides. See article 113 on sewers.

(*v*) *Channels for variable discharge.*—When the channel has to carry a variable volume, it is desirable that the velocity should be nearly constant, or the H. M. D. should be constant, or the wetted perimeter should increase at the same rate as the area of the section. This condition cannot be conveniently secured in earthen channel. This principle is however adopted to some extent in ovoid sewers for which see article 113.

(*vi*) *Channels of irregular sections.*—See Chapter XIII on rivers.

94. *Discharge of channels.*

If the velocity and discharge of an existing channel are required, measure the bed width, side slopes, bed slope, calculate velocity from Kutter's formula and the discharge then is equal to area × velocity.

If the new channel is to be designed, determine the bed slope from the available slope of the ground, determine the type of section from the geological formation of the ground, ascertain the required discharge and decide upon suitable velocity, not too low to allow silt deposit and not too high to cut up bed and sides and then from Kutter's formula, find out the cross section, etc.

Tables given in this chapter and nomograms No. 5 and 6 will be of great help for the purpose.

The general problem in connection with the flow of water in open channels involves four variables.

1. Discharge or velocity. 2. The bed width. 3. The depth of water. 4. The bed slope.

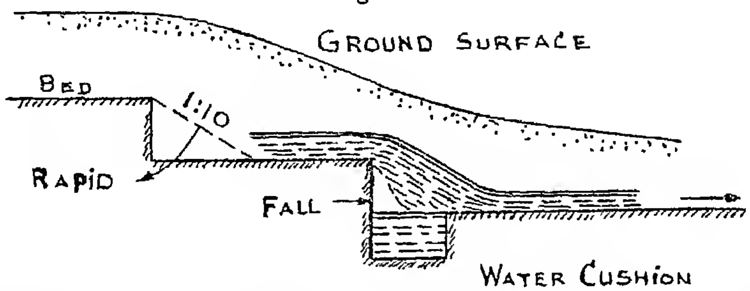
From the nomograms, the required variables can be found; if some variables are given, if necessary, approximate assumption can be made and by trial, correct values can be found.

95. *Falls.*

To design a channel, it is necessary to know the discharge, the geological formation of the soil through which the channel is to be constructed, and the general slope of the ground. The geological formation fixes the velocity within certain limits. It should not be low to allow " silt and weeds " to settle down to the bottom of the channel. It should not be too high to cause the erosion of the bed.

Knowing the discharge and the velocity, economical cross section of the channel is to be worked out; and thereafter the bed slope of the channel is to be found. If the ground slope is steeper than the proposed bed slope, then the channel is to be divided into convenient reaches with suitable falls at the ends of these reaches as shown in Fig. 68.

Fig. 68



If the channel is cut through rocky ground, a rapid is built because it is economical to do so with stone available from the rock cutting.

In mountain streams, one sees vertical falls with water cushions, formed by nature. The depth of the water pool at the foot of the natural fall is a guide to us to fix the depth of water cushion.

On the Ganges canal as first constructed, the falls had their crests at bed level of their upper reach. This led to the reduction of depth of water in the channel for a considerable distance back from the crest of the fall. Consequently velocity increased and the scour produced in the bed of the channel above the falls was so great that it was found necessary to raise the crest of the fall by a weir.

Fig. 69

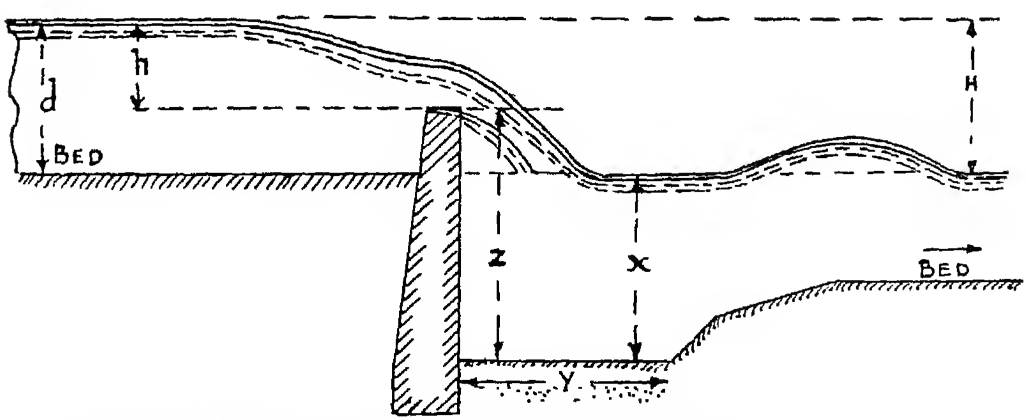


Fig. 69 shows a fall with a weir. To calculate the height to which the weir should be built, find out the velocity in the channel above the fall from Kutter's formula.

$V = C\sqrt{r}\ \sqrt{s}$. Then find out the head ha due to velocity V : $\left(ha = \frac{v^2}{2g}\right)$; the discharge (Q) of the channel is known, also ha is known.

The length of the weir is known. From formula 24, find out h, the depth of water over the weir. Then d — h is the height to which the crest of the weir should be built above the bed of the channel on the upper side of the fall.

The depth of water cushion is given by the formula.

$$X = h + (h)^{\frac{1}{2}} (H)^{\frac{1}{2}} \qquad \ldots\ldots 74.$$

$Y = \frac{4}{3} (Z)^{\frac{1}{2}} h$: (Bombay P.W.D. Handbook, Vol. II : Page 606).

The width of the cushion should be a little more than the width of the channel bed, so that falling water eddies in the cushion and dissipates its energy.

The crest of the weir should have sides so built as to form a notch similar to the cross section of the channel.

Fig. 70

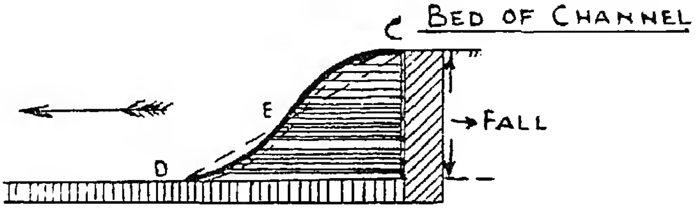


Fig. 70 shows an ogee fall, which consists of a double curve, the object being to deliver the water at the foot of the fall without vertical velocity ; but there will be excessive horizontal velocity which can be checked by widening the channel below the fall. The double chord C D has a slope of about 6 to 1, the chord C E being $\frac{1}{3}$ of chord C.D.

A small weir with its crest raised a little above the bed of the upper channel should be built to prevent erosion of the bed of the said channel.

For channels, carrying small quantities of water ogee falls are cheaper and convenient. For further constructional details, information can be obtained from publication of the Provincial Governments, Irrigation Department.

Mr. Montagu's design for discharges upto 1000 cusecs is the best (Punjab Irrigation Branch). For larger falls and higher discharges per foot run, Poona design is the best, which has been adopted at Tando-Mastikhan Fall on Rohree canal in Sindh.

96. Standing Wave. (Encyclopædia Britannica 9th Edition : Hydromechanics).

Fig. 71

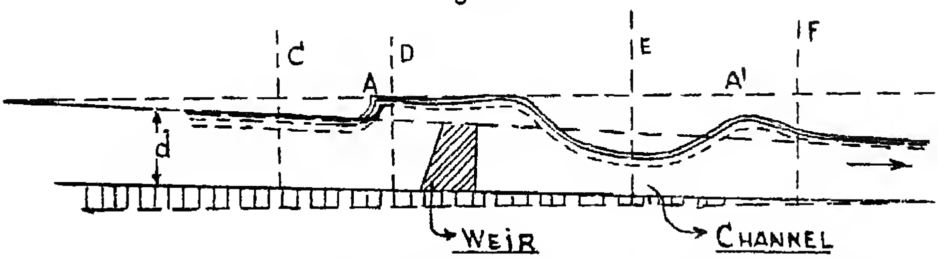


Fig. 71 shows a channel with uniform width and bed slope. A weir is built across it. The weir acts as an obstruction to the flow of water. Let d be the depth of flow in the channel and V the velocity of flow. If the stream has a high velocity relatively to the depth of water in it and d be less than $\frac{v^2}{g}$ and if an obstruction in the shape of a weir is placed on it, the following phenomenon occurs.

As the water flows towards the weir, the velocity decreases and d increases. At a point A, $d = \frac{V^2}{g}$ the water surface tends to become normal to the bed, a standing wave is formed. On the down stream side of the weir, at point E, the velocity is so great and the depth is so small that d may be less than $\frac{V^2}{g}$. As the water moves on, V decreases due to friction against the sides of the channel and d increases ; at point A^1 where $d = \frac{V^2}{g}$ a standing wave is formed.

The primary condition that d be less than $\frac{V^2}{g}$ mentioned above implies that d is less than $\frac{C^2 r s}{g}$ because $V = C \sqrt{r}\ \sqrt{s}$. In broad shallow channels d approximates r. In that case s must be greater than $\frac{g}{C^2}$. In the Kutter's formula, if $C = 100$, then s must be more than ·0032 or the bed slope of the channel should not be less than 16 feet per mile.

Fig. 72

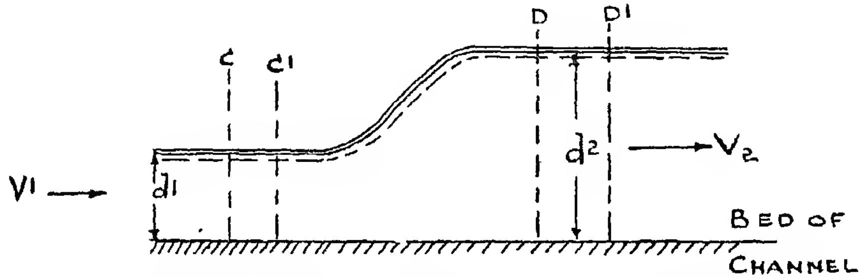


Fig. 72 shows a standing wave on a large scale. The height of the wave can be found as follows :—

Let the mass of water C D move forward and occupy the position C^1, D^1 in time t seconds. At point c, let the depth of flow be d_1 : width of flow be b : area of flow be A_1. At point D, let depth of flow be d_2, width b, Area A_2.

We know, Force = Mass × change of velocity per second. The horizontal change of momentum is $\frac{W}{g}\left(A_1 V_1^2 - A_2 V_2^2\right) t = \frac{w\ b}{g}\left(d_1 v_1^2 - d_2 v_2^2\right) t$:

But $A_1 V_1 = A_2 v_2$ or $b\ d_1 v_1 = b\ d_2 v_2$ or $d_1 v_1 = d_2 v_2$:

Or $V_2 = \frac{d_1}{d_2} v_1 \quad \therefore \quad d_2^2 - d_1^2 = \frac{2}{g} v_1^2 \frac{d_1}{d_2} (d_2 - d_1)$:

$\therefore \quad d_2 + d_1 = \frac{2 v_1^2 d_1}{g\ d_2}$, whence

$$d_2 = \left(\frac{2 v_1^2}{g} d_1 + \frac{d_1^2}{4}\right)^{\frac{1}{2}} - \frac{d_1}{2} \qquad \text{.....(75)}$$

The height of the wave is $d_2 - d_1$.

Fig. 73

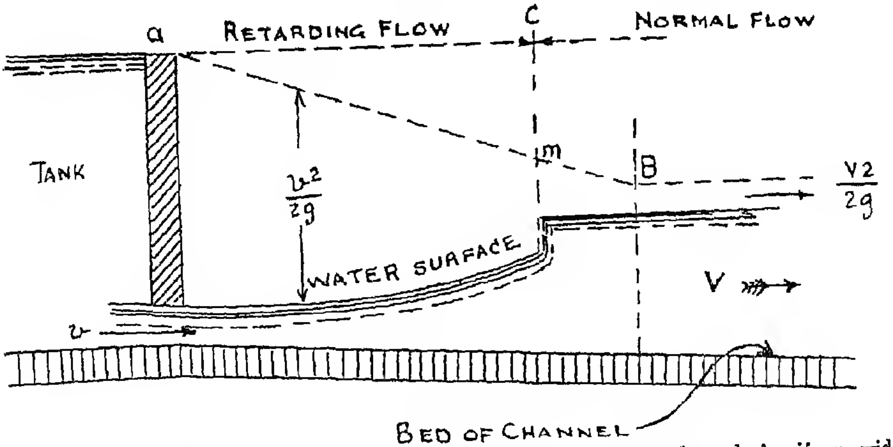


Fig. 73 shows water issuing from a sluice at a high velocity V into a channel of uniform, width. As the water flows on, due to friction against the sides of the channel the velocity V decreases. This phenomenon goes on till a standing wave is formed at a point m due to the fact that energy loss is proportional to V^2 and the increase in depth is only proportional to V.

The line a m B is the total energy line. At point B, the flow becomes of normal type and water moves on in the channel with normal velocity V, due to the bed slope of the channel. The energy line is parallel to the water surface and at a height of $\frac{V^2}{2g}$ feet from the surface. See "The standing wave or Hydraulic Jump", Publication No. 7, Central Board of Irrigation, Simla.

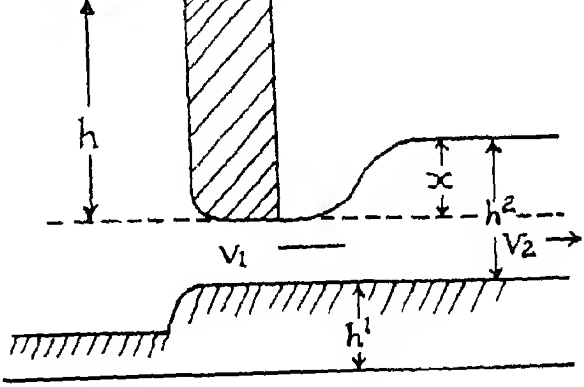
The phenomenon of standing wave is liable to occur at the foot of waterfalls, in the exit channel of sluices, in the vicinity of under water obstructions, when a steep wooden trough tails into a pond of water, on the downstream of a sloping weir, downstream of a contracted water-way, and on the downstream side of bridges discharging in times of flood.

At the foot of the rapid, forming the left flank of the weir across the river Ravi at the head of the Bari-Doab canal, the standing wave, when the floods are passing, is 6 to 8 feet high, not counting the masses of broken water on the crest of the wave. Logs 6 feet in diameter brought down by the flood disappear in the wave.

For more information on standing wave, etc., see Paper No. 126, P. E. C. (1929) and Central Board of Irrigation, Simla, Paper No. 7.

"When a shallow stream moving with a high velocity strikes water of sufficient depth, there is commonly produced a striking phenomenon which has been appropriately called the Hydraulic JUMP." Fig. No. 73 (*a*) explains this.

Fig. 73a



Momentum equation of the Jump : $X = \left(\frac{h^2}{4} + \frac{2h\, v_1^2}{g} \right)^{\frac{1}{2}} - \frac{3}{2}\, h$ = Height of Jump :

Hydraulic Jump is utilized in India in the so-called "Standing Wave" flumes in distributing outlets for irrigation. Mr. C. C. Inglis, Superintending Engineer, Irrigation, Poona, is known to have studied the phenomenon and the results of his experiments can be had from the Government Research Laboratory at Poona.

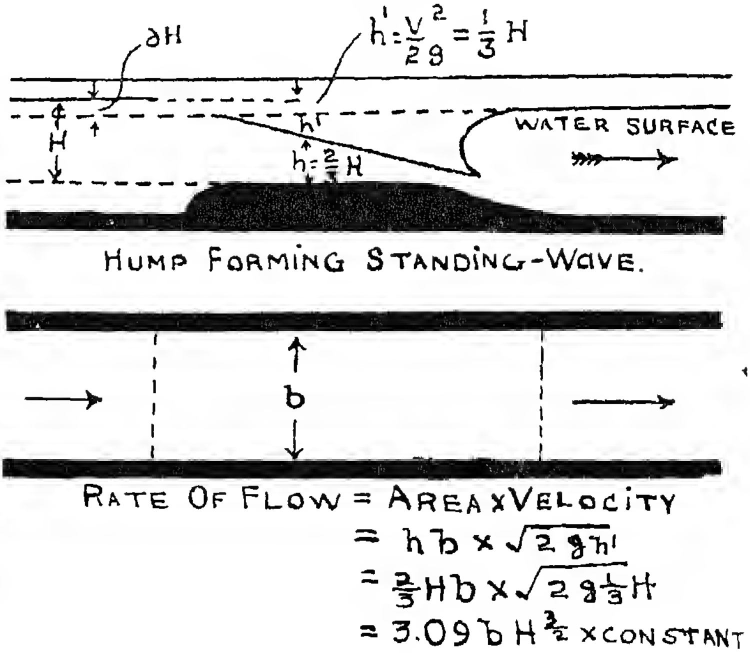
A very comprehensive account of the use of Hydraulic Jump as a mixing device at Kirtland Pumping Station, Cleveland, Ohio, U.S.A., may be found in the Journal of the American Water Works Association, Vol. 17, January 1927 ; on the same subject, See "Novel features of water purification plant for the new water works at Gwalior" by S. T. Prokofief in the Journal of the Institution of Engineers (India), Vol. XI, May 1932 : In this case alum is put in raw water for thorough mixing by means of the device mentioned above.

Fig. 73b shows a standing wave flume, used to measure accurately the flow of water in open channels. A hump is built to form the standing wave. The action is based on two fundamental Laws :—

(*i*) The Law of bodies falling under the effect of gravity, $V^2 = 2gH$, where V is the velocity created by the head H.

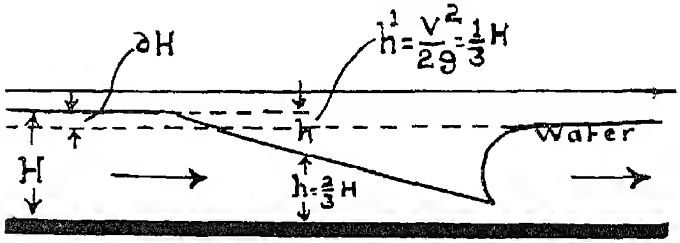
(*ii*) Bernoulli's theorem which interpreted for open channel flows, states that in a frictionless channel, the Sum of Potential Energy, the Kinetic Energy and Pressure Energy remains constant. In an open channel, the pressure energy can be neglected by assuming that all the water in any Section has the same energy as that on the surface and thus potential energy plus the velocity energy remains constant. In other words $H + \frac{V^2}{2g} = K$: This means that any increase in velocity entails a corresponding loss in head or depth. It is, of course, another form of the principle of the Conservation of Energy.

Fig. 73b



The shallow fast flowing stream over the Hump in the channel cannot obviously continue further as its rate of losing energy is much greater than that of deep slow moving stream whose rate of losing energy is usually determined by the gradient of the channel downstream. A change must therefore take place ; as all intermediate depths are unstable, the stream has no alternative but to jump to the higher level. This abrupt transition forms what is called a standing wave. This wave is merely a means of dissipating the difference of energy between the upstream and downstream water-levels.

Fig. 73c



Side Contractions Forming Standing wave

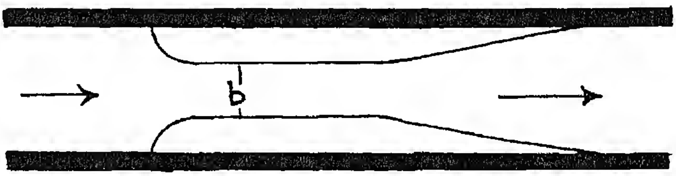


Fig. 73c shows another form of a standing wave flume. Here the channel is constricted sufficiently to create the standing wave. The constricted area is the throat in which the water-surface falls. The fall represents the energy required to create the additional velocity in the throat and if this velocity is V, then the fall in the water surface is represented by $\frac{V^2}{2g}$. Thus on the downstream side of the throat, the velocity will decrease and water surface will regain its previous level.

Rate of Flow = Area × Velocity

$$= hb \times \sqrt{2gh'}$$

$$= \tfrac{2}{3}Hb \times \sqrt{2g\tfrac{1}{3}H}$$

$$= 3{\cdot}09\,bH^{\frac{3}{2}} \times \text{constant}$$

If the amount of constriction is increased, the velocity in the throat increases very rapidly and the depth decreases correspondingly until a point is reached when standing wave is formed: The size of the throat determines the upstream depth upto a certain limit. Water tries to pass through the throat as fast as it can do under the head available and so the water downstream is not impeding the flow in any way. In other words, if the downstream water disappeared, the flow through the throat will not be affected. The water is said to be free flowing.

The discharge is $3{\cdot}09\ bH^{\frac{3}{2}} \times$ constant, thus depending upon the water-surface level on the upstream side. Messrs. Lea Recorder Co. Ltd. of Manchester have devised the size of a throat, relative to the size of main Channel, an apparatus to record the water-surface levels on upstream side of the throat or Hump by means of a float and trace the discharges on a graph-paper automatically by means of a clock mechanism. Such recorders have been put up on sewage disposal works in different parts of British Empire.

97. *Specific energy of a Channel's cross section.* Fig. 74 shows the cross section of a channel with a constant quantity of water flowing through it.

$\therefore\ Q = b\,d \times V$: V varies as d.

The specific energy of the stream at the cross section consists of static energy due to its depth plus the kinetic energy, without reference to any datum height or slope of the channel. (Lewett).

$$\text{Specific energy} = E = d + \frac{V^2}{2g} \qquad \ldots\ldots 76$$

Fig. 74

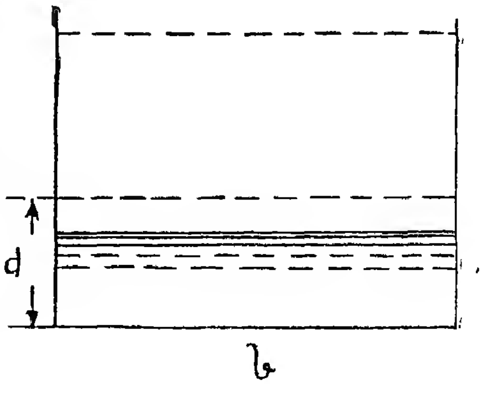
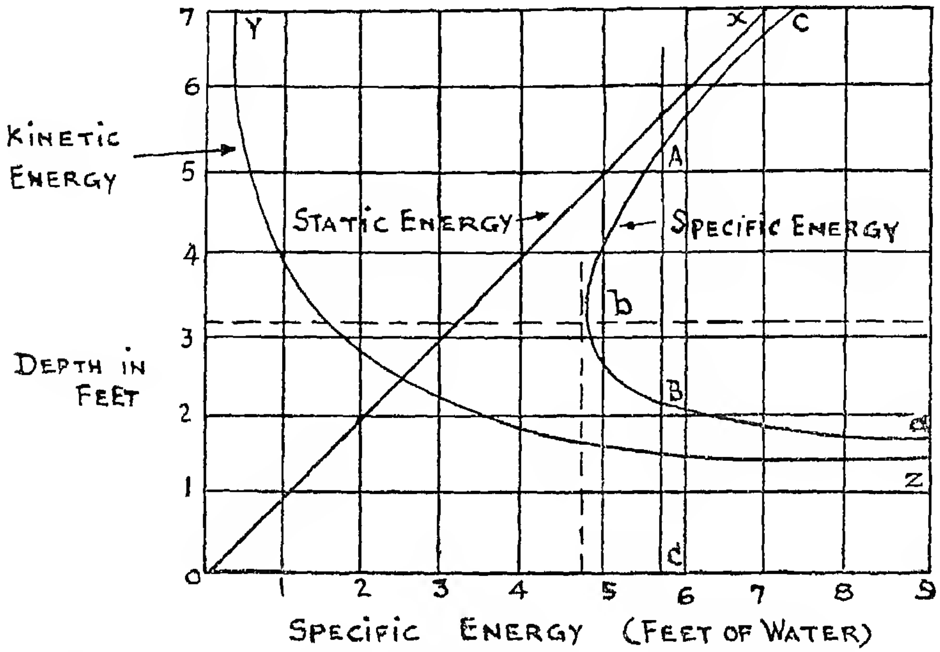


Fig. 75



In Fig. 75, the static energy, kinetic energy and specific energy for the cross section in Fig. 74 have been plotted for a fixed quantity of flow and various depths of stream. The static energy is represented by the straight line ox; the corresponding kinetic energy by the curve YZ. By adding the horizontal ordinates of these curves, the specific energy line a b o is obtained. It will be seen that specific energy at first becomes less as the depth increases, reaching minimum value at point b: beyond the point b, the specific energy increases as the depth increases. The depth at the point b is the depth at minimum energy and is called the *critical depth.* For each value of specific energy to the right of b, there are two depths for the given quantity of flow considered; both of these depths produce the same specific energy.

If a horizontal line is drawn through b, the area above this line is called the area of tranquil flow, the area below is known as the area of rapid flow.

The depth at the minimum specific energy or critical depth is obtained from the following equation, where v is critical velocity.

$$v^2 = gd \qquad \ldots\ldots 77$$

or $\frac{v}{\sqrt{(gd)}} = 1$. This term $\frac{v}{\sqrt{(gd)}}$ is called the Froude number. Hence the minimum energy occurs when the Froude number is unity. The Froude number is a non-dimensional Factor governing the gravity effect. If v is exceeded, the flow is turbulent.

Also depth of a channel for a maximum flow for a given specific energy is the critical depth.

The reader is recommended to go through the paper No. 212 of P E. C. Vol. 26 of 1938 on energy theory of turbulent flow.

TABLE IV.

The following table gives the bottom velocity in a channel which just produces motion in the substances mentioned :—

(P. W. D. Handbook : Bombay, Page 605).

Material.	Feet per Second.	Material.	Feet per Second.
Soft Earth	0·25	Gravel and Coarse Sand	1·0
Fine Clay	0·25	Pebbles 1 inch diameter	2·0
Soft Clay	0·50	Pebbles, egg size	3·0 to 3·3
Finest Clay	0·50	Stones, 3 inches diameter	5·0
Fine Sand	0·70	Boulders, 6 inches to 8 inches diameter	6·6
Coarser Sand	0·80	Boulders, 12 inches to 18 inches diameter	10·0

TABLE V.

Relation between mean velocity, hydraulic mean depth and erosive power of a stream.

(P. W. D. Handbook : Bombay, Page 605).

	There is no SCOUR in a channel of H.M.D.	Until a mean Velocity is reached of.
	Feet	Feet per second
Fine Silt	1·0	0·40
	2·5	0·70
	5·0	0·90
	10·0	1·50
Heavy Silt and Fine Sand	1·0	0·9
	2·5	1·5
	5·0	1·75
	10·0	2·25
Coarse Sand	1·0	1·75
	2·5	2·25
	5·0	3·0
	10·0	3·5
Small Pebbles (size of Peas) and Gravel	1·0	2·25
	2·25	3·0
	5·0	3·5
	10·0	4·5
Large Pebbles (Hen's Egg size) and Coarse Gravel ..	1·0	5·0
	2·5	6·0
	5·0	7·0
	10·0	9·0
Large Stones	1·0	15·0
	10·0	23·0

TABLE VI.

Sectional areas and Hydraulic mean depths of earthen channels. (Side slopes ½ to 1.)

Bed width.	Depths.																			
	1		2		3		4		5		6		7		8		9		10	
	Area	H.M. Depth.	A	R	A	R	A	R	A	R	A	R	A	R	A	R	A	R	A	R
2	2·5	·58	6	·93																
3	3·5	·67	8	1·07																
4	4·5	·72	10	1·18																
5	5·5	·76	12	1·27	19·5	1·67														
6	6·5	·79	14	1·34	22·5	1·77	32	2·14												
7	7·5	·81	16	1·39	25·5	1·86	36	2·26												
8	8·5	·83	18	1·44	28·5	1·94	40	2·36	52·5	2·74										
9	9·5	·84	20	1·49	31·5	2·06	44	2·45	57·5	2·85										
10	10·5	·86	22	1·52	34·5	2·07	48	2·53	62·5	2·95										
12	12·5	·89	26	1·58	40·5	2·17	56	2·67	72·5	3·13	90	3·54								
15	15·5	·90	32	1·64	49·5	2·28	68	2·84	87·5	3·34	108	3·80								
18	18·5	·91	38	1·69	58·3	2·37	80	2·97	102·5	3·51	126	4·01								
20	20·5	·92	42	1·72	64·5	2·41	88	3·04	112·5	3·61	138	4·13	164·5	4·61						
25	25·5	·93	52	1·76	79·5	2·51	108	3·18	137·5	3·80	168	4·37	199·5	4·91	232	5·41				
30	30·5	·95	62	1·80	94·5	2·57	128	3·29	162·5	3·95	198	4·56	234·5	5·14	272	5·68	310·5	6·19	350	6·68
40	40·5	·96	82	1·84	124·5	2·67	168	3·43	212·5	4·15	258	4·83	304·5	5·47	352	6·08	400·5	6·66	450	7·22
50	50·5	·97	102	1·87	154·5	2·72	208	3·53	262·5	4·29	318	5·01	374·5	5·79	432	6·36	490·5	6·99	550	7·60

Table VII.

Velocities calculated by Kutter's formula

N = ·025

Hydraulic Mean depths.

Slope 1 in	·5	·8	1	1·5	2	2·5	3	3·5	4	4·5	5
100	3·7	4·6	5·6	7·6	9·3	10·9	12·3	13·7	15·0	16·2	17·4
150	3 0	3·7	4·6	6·2	7·6	9·9	10·1	11·2	12·3	13·3	14·2
200	2·0	3·3	3·9	5·3	6·6	7·7	8·7	9·7	10·6	11·5	12·3
300	2·1	2·6	3·2	4·4	5·4	6·3	7·1	7·9	8·7	9·4	10·1
400	1·8	2·3	2·8	3·8	4·6	5·4	6·2	6·9	7·5	8·1	8·7
500	1·6	2·0	2·5	3·4	4·2	4·9	5·5	6·1	6·7	7·3	7·9
750	1·3	1·7	2·0	2·7	3·4	4·0	4·5	5·0	5·5	5·9	6·4
1,000	1·3	1·7	1·8	2·4	2·9	3·4	3·9	4·3	4·8	5·1	5·5
1,500	·9	1·2	1·4	1·9	2·3	2·6	3·1	3·4	3·7	4·1	4·5
2,000	·8	1·0	1·2	1·7	2·1	2·4	2·8	3·1	3·4	3·7	3·9
3,000	·7	·8	1·0	1·4	1·7	2·0	2·2	2·5	2·7	3·0	3·2
4,000	·6	·7	·9	1·2	1·5	1·7	2·0	2·2	2·4	2·6	2·8
5,000	·5	·5	·9	1·0	1·3	1·5	1·7	1·9	2·1	2·3	2·5
6,000	·5	·4	·7	·9	1·2	1·4	1·6	1·8	2·0	2·1	2·3

The velocities for intermediate slopes or depths might be found approx., or depths might be found approx., by adding proportionate parts of difference to the velocities given by the table for the next lower slope or depth.

V = mean velocity in feet per second. R = Hydraulic mean depth in feet. S = slope of surface = fall of surface divided by that Length : N = coefficient of friction or modulus of rugosity of sides.

Table VIII

Table for "Best Discharging" Channels.

Side slopes.	Depth of water in channels.	Width of base of the channel.	Width of the top of the channel.	Hydraulic mean depth of the channel.
	Square root of the area of the water way of the channel multiplied by			
Semicircle	·798	0	1·596	·399
0 to 1	·707	1·414	1·414	·354
½ to 1	·759	·938	1·697	·379
¾ to 1	·748	·675	1·996	·374
1 to 1	·740	·613	2·093	·370
1½ to 1	·689	·417	2·484	·345
2 to 1	·636	·300	2·844	·318

TABLE IX.

Values of S and $\sqrt{S}$

To find $\sqrt{S}$ for a steeper slope, look out a slope 4 times as flat and multiply $\sqrt{S}$ by 2. Thus for 1 in 50, $\sqrt{S}$ is ·07071 × 2 = ·14142.

Slope 1 in	S	$\sqrt{S}$	Slope 1 in	S	$\sqrt{S}$
100	·010	·1	350	.002857	.05345
150	·006667	·08165	400	·0025	·05
200	·005	.07071	460	·002174	·04663
250	·004000	.06325	2500	..	·0200
300	·003333	·05774	2700	..	·01925
500	·002	·04472	3000	..	·01826
550	..	·04264	3300	..	·01741
600	..	·04083	3600	..	·01667
650	..	·03922	4000	..	·01581
700	..	·03780	4500	..	·01491
750	..	·03652	5000	·0002	·01414
800	..	·03536	5500	..	·01349
900	..	·03333	6000	..	·01291
1000	·001	·03162	6500	..	·01240
1100	..	·03015	7000	..	·01195
1200	..	·02887	7500	..	·01155
1300	..	·02774	8000	..	·01118
1400	..	·02673	8500	..	·01085
1500	..	·02582	9000	..	·01054
1600	..	·02500	10000	..	·0100
1800	..	·02357	20000	..	·0071
2000	·0005	·02236			
2200	..	·02132			
2400	..	·02041			

Examples :—1. A small earthen channel has a bottom width of 6 feet, side slopes ½ to 1 : depth 3 feet : Bed slope 1—1000. Find the velocity and discharge ?

See Table No. I : N = ·025 : From Table No. VI, Area = 22·5 sq. feet. H.M.D. = 1·77 ft. : From Table VII, $V = \frac{2{\cdot}4 + 2{\cdot}9}{2}$ = 2·65 Feet per second : Q = 22·5 × 2·65 = 59·62 cusecs :

Also see Nomogram No. 5 (Fig. 76) : C, the coefficient in Kutter's formula is equal to 62 : $V = c\sqrt{r}\sqrt{s}$ $= 62 \times 1{\cdot}77 \times \frac{1}{1000}$ = 2·604 ft. per second : Q = 22·5 × 2·604 = 58·6 cusecs :

2. An earthen channel has to discharge 500 cusecs with a velocity of 3 feet per second. Bed slope 1—2500, the available slope of the ground surface : side slope half horizontal to one vertical. Kutter's coefficient of rugosity N = ·025 ; find the depth and bottom width.

(*a*) In Plain ground : From Table III, the limiting depth, for a velocity of 3 ft. per second, to be under 8 feet : The area of section will be $= \frac{500}{3}$ = 166·66 sq. ft. See Table VII. For slope 1—2500 and Vel. 3 ft. per second, Hydraulic mean depth r = 4 feet.

See Table VI.

R. ft.	Bed-width : ft.	Depth : ft.	Area sq. ft.
3·43	40	4	168
3·95	30	5	162·5
4·15	40	5	212
4·37	25	6	168
4·61	20	7	164

KUTTER'S FORMULA FOI
EXPRESSION $v = C.\sqrt{rs.}$

KUTTER'S FORMUL

N. IS A COEFFICIENT VARYING FR(

'om Nomogram 5 (Fig. 76), R = 4 : c = 74 :

$$V = c \sqrt{r} \sqrt{s} = 74 \times 2 \times \cdot 02 = 2 \cdot 96 \text{ feet per second.}$$

'om Nomogram No. 6 (Fig. 77), also V comes to 3 ft. per second.

(*b*) If the earthen channel is to be built in the sloping sides of earth Hills, as in the case of Power-Duct , Khandala G.I.P. Rly., for Tata Hydro-Electric Scheme, the width should be less and depth more, proportionately ; under.

From Table VI, Bed width = 16 feet : Depth = 8 feet.

$\sqrt{R} = 2 \cdot 17$: Area of channel = 160 sq. ft. side slopes $\frac{1}{2}$ to 1 : C = 78 : $V = c \sqrt{r} \sqrt{s} = 78 \times 2 \cdot 17 \times \cdot 02 = 3 \cdot 38$ Ft./s. If the subsoil water level is 8 ft. below the ground surface, then reduce the depth and increase the 'idth.

3. In an earthen channel with vertical sides, bed width 70 feet and depth 5 feet, the surface velocity at the entre is 3 feet per second. N = ·025 : What is the mean velocity Vm, in the whole channel ?

See last 2 paras of article 89. The mean velocity Vc on the central vertical line of the channel = Vs × ·89 = 2·67 Ft./Sec., Vs being surface Velocity at the centre *viz.* 3 Ft./Sec.

The mean velocity Vm in the whole channel is = Vc × ·94 = 2·67 × ·94 = 2·52 ft. per second.

Vote.—*i.* *Nomogram* No. 5 (*Fig.* 76), gives values of c *for different* values of n, s, and R. Then V can be found from the formula $V = c \sqrt{R} \sqrt{S}$.

ii. Nomogram No. 6 (Fig. 77), gives relative values of V, n, R, and S : Based on Kutter's formula.

From the intersection of the H. M. Radius R and n follow the vertical line to the intersection of S and V ; or from the intersection of slope S and V, follow the vertical to the intersection of R and n.

(U. S. Dept. of Agriculture Bulletin 852 : 1920).

CHAPTER XII.

Irregular open channels.

99. Channel with varying cross section.

When the flow is variable, the loss of head from resistances is the same as in a uniform stream, provided the change of section is gradual and the length of the channel is short so that the velocity and hydraulic radius change very little. The formula $V = C\sqrt{r}\sqrt{s}$ applies to a variable stream of a given length, provided the cross sections at the ends are similar and velocities equal, and the fluctuations in the cross sections between the two ends are moderate. Though 's' varies from point to point, it is the total fall from end to end that counts. It has been ascertained that a variable stream is less efficient than a uniform stream of the same mean section or in other words it must have a great total fall in order to carry the same discharge.

100. Channels with uniform cross section, uniform slope but variable flow.

This is a case of sullage drains and sewers in towns and cities and forms the subject matter of Chapter No. XV.

Inundation canals also come under this head.

101. Channels with varying cross sections, varying bed slopes and variable flow.

This is a case of mountain streams, torrents and rivers. In Himalayas the streams carry during dry weather, water due to melting snows and their discharge can be easily measured by forming a uniform channel in a suitable length of the stream.

The torrents carry flood water during rains and the discharge can be calculated from flood marks in a straight length of uniform section and uniform slope.

The case of rivers forms the subject matter of a separate chapter No. XIII.

102. General remarks (Bellasis).

(*i*) *Bends* :—At a bend there is a 'set of streams' towards the concave bank, the greatest velocity being near that bank; and there is a raising of water level at that side so that the water surface has a transverse slope.

Fig. 78

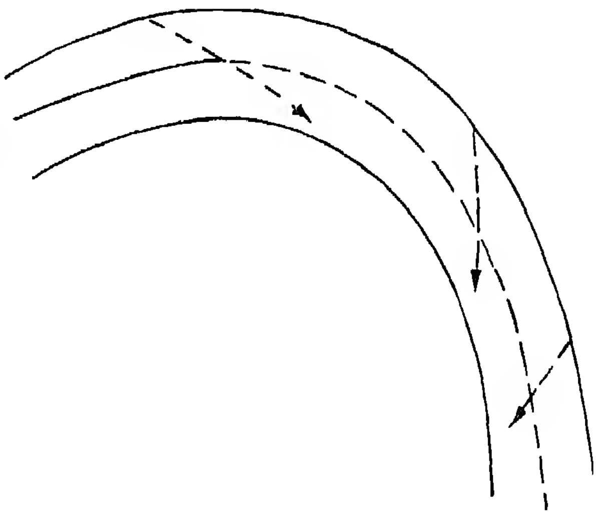


Fig. 78. There is also deepening in the concave bend and shoaling at the convex side. This is not all due to the direct action of the centrifugal force. The high water level at the concave bank due to centrifugal force gives a greater pressure and tends to cause a transverse current from the concave side to the convex side. This tendency is in the greater part of the cross section, resisted by the centrifugal force, but the water near the bed and sides has a low velocity, the centrifugal force is therefore smaller and transverse flow occurs. The solid material is thus rolled towards the covex banks and it accumulates there because the velocity is low. To compensate for the low level currents towards the convex bank there are high level currents towards the concave bank.

In fig. 78, the dotted line shows the direction of the strongest surface current and the arrows show the current near the bed. This explanation is due to Thomson and has been confirmed by him experimentally.

(*ii*). *Changes of section* :—Take the case of a bridge pier in a stream, or a spur at the bank of a river. The section *area* is decreased and velocity increased. Hence scour takes place alongside the pier due to increased velocity and downstream of it due to eddies, leading to the formation of holes in the bed.

In a short deep recess in the bed or bank of a stream, or downstream or an obstruction, if it is large enough to cause dead water, there is generally a rapid deposit of silt, but not where strong eddies occur.

(*iii*). For effects of change in the discharge of a channel, for effects of alterations in a channel, effect of a weir or raised bed, see professional papers issued by the Punjab Government Irrigation Branch Secretariat from time to time ; also Bellasis' Hydraulics, Chapter VII complete.

NOTE :—Figures 79 to 90 will appear in 2nd Edition of this book.

(*iv*). *Surface curve* :—In a given channel with a given discharge, there is only one curve of heading up and one of drawing down, whatever the cause of variable flow may be. Practically, the curve extends to a limited distance beyond which no change in the natural water surface is perceptible. The form of the curve is not exactly known.

V. *Variable flow* :—In uniform channels, 'natural flow' and 'uniform flow' have the same meaning.

Fig. 91

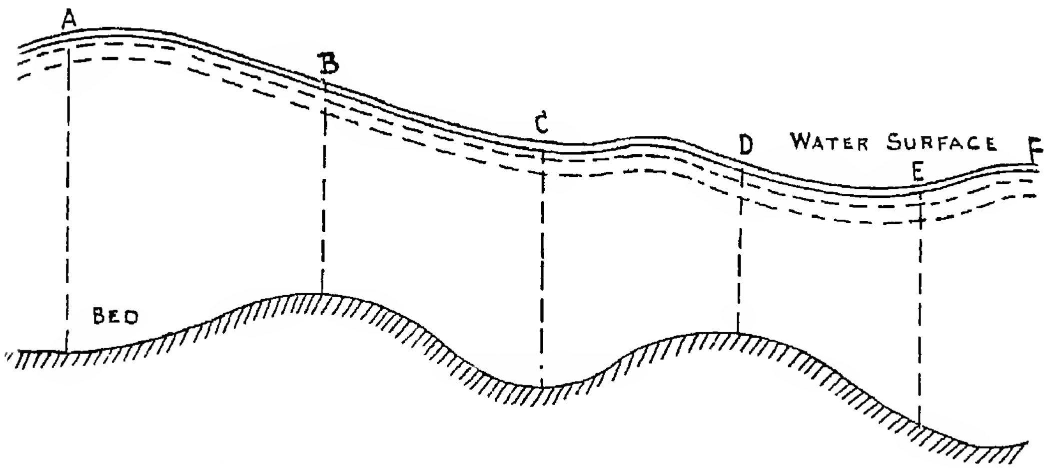


Fig. 91. Shows a variable channel in different lengths and the water surfaces also in different lengths, according to discharges. The water surfaces are termed natural surfaces. The bed consists of pools and rapids. In submontane districts at the foot of Himalayas in India, the rivers and torrents assume the section shown in fig. 91. Some times the water heads up and in some places it draws down or these may counteract each other and the flow becomes natural.

The surface slope is given by the formula $V = C\sqrt{r}\sqrt{s}$, where 's' is the surface slope. At any selected point, let B be the width of water surface and d the mean depth. Then roughly $Q = A\,V = B\,d\,c\,\sqrt{r}\sqrt{s}$. The surface velocity can be ascertained by means of a float. Area can be worked out and so 'r' can be found, thence 's' is obtained.

If from any cause, heading up or drawing down occurs at point F, the water surface will undulate, approaching the natural surface towards A. The greater the depth of water in the channel, the less the effect of inequalities in the bed. A stream at high water has a fairly uniform surface slope and at low water forms a succession of pools and rapids.

A stream may be so irregular in plan and section that the direction of current is not parallel to what may be seen to be the axis of the channel and the water surface far from level across. The irregularities are due to curves, obstructions, specially at low water.

In a variable stream, a short length L can be found in which the flow is uniform. Slope can be ascertained & so the velocity by observations. Velocity can be also calculated from $V = C\sqrt{r}\sqrt{s}$ Thus the discharge can be found.

VI. *Simple waves :—*

Fig. 92

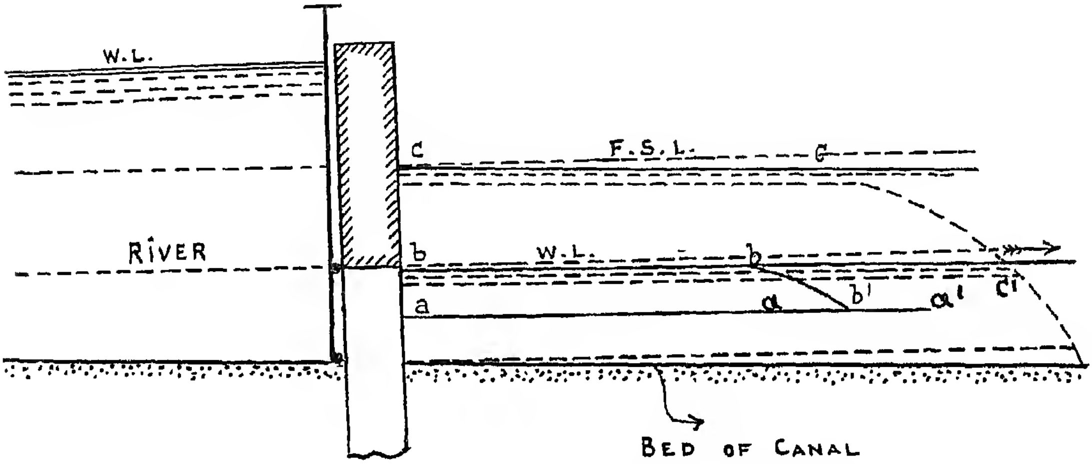


Fig. 92 shows a canal with a regulator with sluice gates. Suppose the canal is dry, when one sluice gate is opened, the water line in the canal is aa. When more sluices are opened, the water line is bb and a wave bb^1 travels along the water surface aa^1.

When more sluices are opened, the water surface is cc and a wave cc^1 travels along the lower water surface.

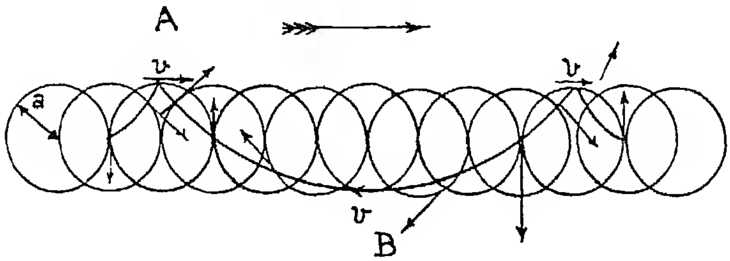
The canal department has fixed up gauges at different points of the canal. The readings which must be very accurate are taken at stated intervals. The surface curve of the wave, which is convex in the above case, can be plotted from these gauge readings. When the supply of water to the canal is regularly closed, the water surface in the canal falls and concave surface waves are formed on the outside of the regulating gates.

The length of such waves is generally very long.

The River Indus, biggest in India, rises in flood during the rainy season of June to September. The flood water level being higher than the water level in a tributary travels in the form of a wave along the water surface of the tributary in the opposite direction. It is very interesting to see this phenomenon.

We will now say something about the surface waves in the channel where the depth of water is very great. The transmission of a surface wave is brought about by a local circulation of the surface water (Fig. 93). Each particle of water on the surface describes circles as shown in the figure 93.

Fig. 93



If V = velocity of wave
L = length of wave.

$$V = \sqrt{\frac{gL}{2\pi}} \quad \text{(Lewett)} \ldots\ldots 78.$$

Waves are formed where a river enters a sea. Tides in a sea enter the river through an estuary. Such subjects are dealt with fully in treatises by Marine Engineers on "Docks & Harbours."

Fig. 94

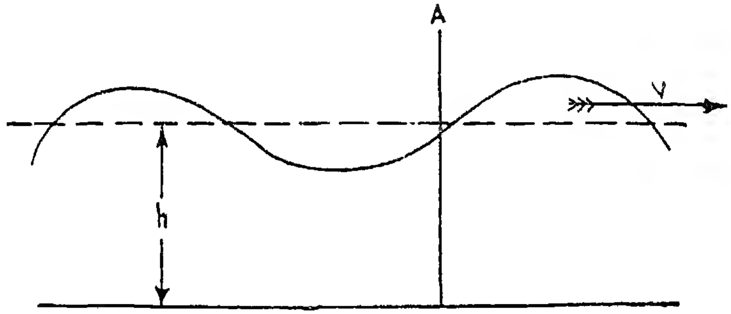


Fig. 94 shows a surface wave in shallow water. Here the wave is transmitted by the local circulation of particle of water near the surface.
Let V = velocity of the wave h = mean depth of water.

$$V = \sqrt{gh} \quad \text{(Lewett)} \ldots\ldots 79.$$

In high storage reservoirs across flat valleys, where the fetch of the waves is very long, the effect due to wave action is specially pronounced.

$$h = 1.5\sqrt{f} + (2.5 - 4\sqrt{f}) \quad \ldots\ldots 79a.$$

h = height of waves in feet : f = fetch of wind in miles (Stevenson).

The force of the wave when it breaks against a masonry wall is = m × v

m = mass of water in lbs.

v = velocity in feet per second.

If the maximum height of the wave is 6 feet, the stability of a masonry dam is calculated for a depth of storage 3 feet above the full reservoir level. This is the usual practice.

(*vii*). Flow in expansions in open channels.

See P. E. Congress paper No. 236 by A. R. Thomas, B.Sc., I.S.E., Central Irrigation and Hydrodynamic Research Station, Poona. This is worth reading.

CHAPTER XIII.

RIVERS

103. *Preliminary* :—Rivers are irregular natural channels. India is lucky to have good many rivers, which through canals irrigate large tracks of country and make India pre-eminently an agricultural country.

The rivers in the North and Northwest parts of India are snow fed from the Himalayas and therefore perennial. Those in the Central parts and Southern parts depend for their water supply on rains, spring water and seepage. During Summer rains, they bring down large volumes of flood-water and innundate large areas of surrounding country. The river Jumna rose in highest flood in 1924 and the river Indus, fed also by five tributaries of Punjab, was in very high flood in the Summer of 1948 and its discharge at Sukkur was 800,000 cusecs, wiping out about 200 villages.

A river is a natural channel irregular because the discharge varies with variation in section, bed slope, and velocity at different times of the year. In plains, due to obstacles in the flow of water, the course is winding with diminution in bed slope and velocity. The solid matter detached from the bed and banks is carried along as silt and sand, depositing where the velocity diminishes. During heavy floods the river makes fresh channels for itself where it enters the Sea, resulting in the formation of "deltas" of rich alluvial soil, wherever the banks approach each other the section of waterway is contracted, the water heads up to produce the velocity necessary to carry the discharge through the contracted section. Hence the surface slope on which the velocity depends, is not generally parallel to the bed slope.

A river, specially at low water, may be a series of separate streams, with numerous junctions and bifurcations. This is the case with Indus at Mianwali on its left bank and Isakhel on right bank in west Punjab. The overall width of the bed is about 10 miles which is a fine sheet of water when the river is in flood. Since a small cross section tends to cause scour and a large one silting, it follows that every stream tends to become uniform in section, to destroy obstructions, to assume a constant slope and to become curved in such a way that its velocity will suit the soil through which it flows.

The river Jamna was in highest flood in 1924, at Delhi surpassing the previous floods on records.

104. *River Discharges.* (Punjab Engineering Congress 1945).

Instruments and appliances used by Engineers nowadays, to calculate the velocity and discharge through channels are described in chapter XVI. Here only mention will be made of the same.

(*a*) Selection of discharge site :—

The river should have a straight reach with regular stream line flow and no bends. The section should be regular and deep, with stable bed and sides and suitable for regular gauge discharge relationship. The site should be easily accessible and free from cracks, free from projections in bed or side, free from rapids, falls, etc.

(*b*) Spacing of sounding points :—

The distance between sounding points depends upon the width of the stream, profile of the bed and accuracy required : see rules of Punjab Government on this subject. The Principle to be followed is that in the portion of the section where the discharge is concentrated, the segments must be nearer, and farther apart in the slack portion so as to have segments of more or less equal discharges.

(*c*) Procedure and equipment :—

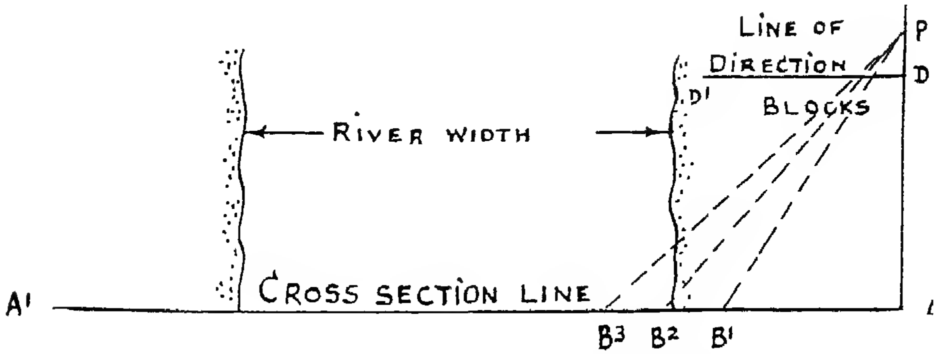
At each discharge site the following is required. A boat with an anchor, sounding rods, current meter, stop watch, a torpedo float with cotton cord, Pocket sextant, a levelling instrument, and flags.

Cross section line is marked by three flags, 200 feet apart on each bank. In case of swift currents, the boat drifts down and flags 25 feet apart are fixed for a requisite distance downstream of the cross section line.

In case of narrow streams, say up to a width of 1000 feet as is the case with Punjab rivers in winter, the section line is marked across the stream by means of a wire, on which pendants are hung at sounding points. The boat is held by another wire, stretched a little upstream, so as to have the rack and pinion under the pendants.

When streams are too wide for stretching a wire rope, the sounding points are located by pivot point method which is described in Figure No. 95.

Fig. 95



From point A on the cross section line on the bank, a line A P at right angles is drawn and from a point D on it, a line parallel to the section line is drawn. The ratio PD : PA is generally kept 1 : 5 and the length of A P is 1000 feet or upto half the width of the river. On the line D D′, points say 20 feet apart are marked and rays from P passing through these points intersect the section line at B_1 B_2 B_3 each 100 feet apart, as is evident from principles of similar triangles. The point P is called the 'pivot point'. All points are marked by cement concrete blocks with holes for flages. The use of theodolite for marking points and lines is advisable.

For rivers wider than 2000 feet, pivot point layouts are made on both banks, for work by two different parties from the two banks.

The section of the river being divided into segments, the boat is taken to a station by a wire rope (power launch may be necessary in case of rough water and very high velocity) or by a pivot point method. Then the soundings are taken and the section of the river is plotted on paper.

By means of a current meter, the velocity is ascertained by lowering it in water to a point 0 6D above bed, D being total depth : This is the mean velocity (not applicable to hilly torrents) of the segment of river section. If a surface float is used, this mean velocity is 0.89 of the surface velocity, see article 89. Knowing the areas of the segments and the mean velocity of the same, the discharges are worked out and the total is the discharge of the river. For surface floats, etc. see chapter No. XVI.

In the absence of a current meter, velocity is ascertained from Chezy's formula $V = C\sqrt{r}\,\sqrt{s}$.

Find out 's', the slope of the channel by taking bed or water surface levels. The hydraulic mean depth r can be worked out from the cross section of each segment into which the river is divided ; for value of the coefficients C, take Kutter's formula (see article 88) and value of n, the coefficient of rugosity as varying from .02 to .03, depending on the condition of the river.

At Sidhani in the river Ravi 800 feet wide, 6 to 10 feet deep, the value of n was found to be 0·10, the case being specially investigated by E. S. Bellasis, I.S.E. (1906). This low value is probably due to effect of eddies from the sides. The following figures may be of interest :

	Value of n
River Ohio at Point pleasant	·021
,, Seine at Paris	·025
,, Mississipi	·027
,, Rhine at Basle	·030

Thus having worked out the values of V for different segments, whose areas are also known, the total discharge is found.

While working out the discharge of river Sutlej in winter of 1909, on the north side of Simla, inflated skins of Buffaloes were used as boats by the writer.

The above method of calculating the discharge by Kutter's formula is also applicable in the case of rivers, flowing between two high cliffs, where the great velocity will not permit of boats being used. If a cross section of such a stream is taken in dry season and plotted on paper, the high flood marks on the sides of the river can also be marked on this cross section and discharge worked out. This method is very useful in the case of large rivers like Indus, Ganges and Jumna while in floods.

If Q be the discharge with measured velocity, and Q_1 be the required flood discharge, then

$\frac{Q^1}{Q} = \frac{c\,A_1\,\sqrt{r_1}}{c\;A\;\sqrt{r}}$; this supposes that bed slope remains constant during floods but it is not so. The bed level is lowered by scour.

This method applies to different segments of the river while calculating discharge.

If there is a bridge across a river, the boats can be tied to the bridge at different points and hydraulic observations made.

The Public Works Department in India have put up gauges in rivers at suitable places and have worked out diagrams of discharges for different depths of flow in the river. See chapter No. XVI on Hydraulic observations.

To find out the flood discharge in a river there is another way of solving the problem. Calculate the catchment area, and the rainfall at the particular period.

If there be a weir across a river, the discharge over the weir gives the discharge of the river. This is described in chapter XVI.

105. *Bends in rivers.*

Fig. 96

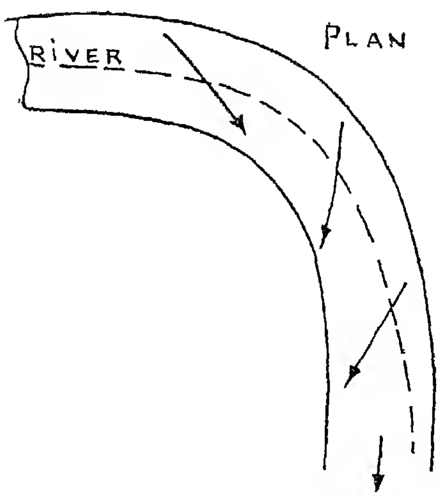
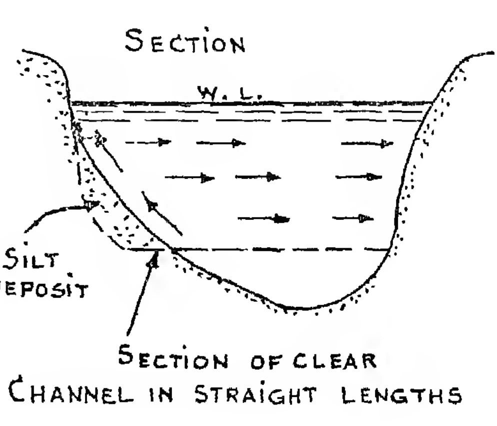


Fig. 96A



In article 91, mention is made of bends in artificial channels. Here we talk of bends in rivers. At a bend there is a ' set of the stream ' towards the concave bank, the greatest velocity being near that bank ; and the water level rises there and the water surface has a transverse slope. (Fig. 96A). There is a deepening near the cocave bank and shoaling at the opposite bank, due partly to the action of centrifugal force. The high water level at the concave bank, due to centrifugal force, gives a greater pressure and tends to cause a transverse current from the concave bank to the convex. The water near the bed and sides has a low velocity and the centrifugal force is small and the transverse flow occurs. Solid material is therefore rolled to the convex bank and it accumulates there because the velocity is low. To compensate for the low level current towards the convex bank, there are high level currents towards the concave bank. The directions of the currents are shown by arrows in Fig. 96A. In Fig. 96 the dotted line shows the direction of strongest surface current and the arrows, the current near the bed. This explanation is due to Professor James Thomson and has been confirmed by him by experiment.

A bend always tends to increase, the greater velocity and greater depth near the concave side help to increase each other. The concave bank is worn away by erosion near the bed, cracks, falls in and is washed away. Some of the large rivers in India flowing through alluvial soil, sometimes cut away at bends, hundreds of acres of land, together with trees, crops and villages standing thereon.

An example of this kind is found in the town of Deragazi Khan on the right bank of Indus, which town was washed away by the river in the end of nineteenth century not withstanding the best efforts of eminent engineers to save the town.

106. *Regime of rivers.* A river is said to be in a state of ' regime or stability ' when its form changes but little from year to year, owing to variations of discharge and consequent erosions and silting which occur at different seasons of the year, a condition of permanent stability is difficult of attainment ; and this is specially the case with Indian Rivers, which have generally sandy beds and which are subject to heavy floods. There is thus a scope for river improvements, which consists in the protection of banks, the prevention of inundations and the removal of obstructions. This subject is dealt with in manuals on irrigation works.

For River training and control see paper No. 275 by A. M. R. Montagu—P. E. Con. Volume (1945).

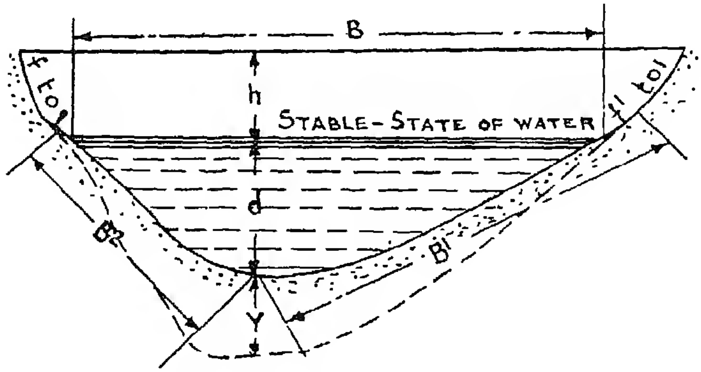
107. *Miscellaneous.* (*i*) *Scour in the bed of rivers* :—There is a certain stage of water level in the river at which no scour takes place and the bed is stable. This stage can be determined by observation or calculation. When the water rises above this level, scour takes place. A mathematical relationship which exists between the rise of water level and the corresponding depth of water is established very ably by Sirdar Karnailsingh and Sirdar Gurdialsingh, engineers on E.P. Rly (see paper No. 254 P. E. Congress 1942.).

It is clear that scour can occur only when the mean velocity in the whole cross section rises as a result of the rise in water level. Assuming the slope of water surface to remain constant, the velocity can only increase by increase in the hydraulic mean radius *i.e.* in the ratio $\dfrac{\text{Area}}{\text{wetted perimeter}}$. $\dfrac{A}{P}$

This can only happen if ' A ' increases at a faster rate than ' P '. It is thus clear that under equilibrium conditions the cross section of the channel will tend to assume a form in which its perimeter is a minimum for a given area, to achieve which the bed of the channel after scour must assume a curved form.

This is explained in Fig. 97.

Fig. 97



A rise 'h' in the water surface has caused a scour 'Y'.

(See Journal of the Institution of Engineers (India) Vol. 26, No. 3, March 1946, page 18. Hardinge Bridge on Ganges at Sara.)

The probable scour in a restricted section can be estimated by the following formula,

$$\frac{r}{r_1} = \left(\frac{W_1}{W}\right)^{\frac{2}{3}} \quad \text{.............................. 97b.}$$

r = Hydraulic mean depth in Regime conditions.

r_1 = do. do. in worst conditions.

W = Width of the stream in regime condition.

W_1 = do. do. in worst condition.

Knowing r_1, the worst depth can be found; this deducted from the H. F. Level will give the lowest point of scour.

(*ii*) *Obstructions in Rivers*:—If a weir is built across a river, it becomes an unnatural obstruction in the free flow channel. The water level upstream of the river, rises considerably and a pocket is formed below the top of the weir on the upstream side. In this pocket, the velocity of flow is reduced and silt deposits. This actually occurs in front of the Barrage recently built across the river Indus at Sukkur in Sindh. When the water level falls, the velocity of flow on the upstream side decreases and silt deposit takes place resulting in the bed slope becoming flatter. Some rocky islands out-crop in river Indus, right in the middle, and scour is noticed on the upstream of the islands and silt deposit on the downstream. When a rocky spur projects from one side into the river, severe eddies are formed on the downstream side of the spur and it is very interesting to watch the flow during flood season and see how the boats in the river Indus avoid going near the dangerous eddies. In the case of Bridges across large channels, the bed upstream of the bridges is found scoured for miles to a depth, sometimes one to two feet below the masonry floors of the bridges, which are left standing up and forming, in fact, submerged weirs. This alone proves the difficult nature of the problem. On this difficult and vast subject see "Minutes of proceedings, Institution of Civil Engineers Vol. CX iii" and several papers issued by the Government of India, Irrigation Department, Simla.

(*iii*) *Development of Rivers*:—The present Government of India are of the opinion that the development of river valley projects is of a basic and most fundamental importance. The Tungbudra in Madras, the Bakra dam in East Punjab, the Damodar Valley in Behar and West-Bengal and the Hirakud dam in Orissa have been taken in hand. These schemes were prepared years ago but the British were slow in taking up their execution.

CHAPTER XIV.

Canals, Waste Weirs and Barrages.

108. Preliminary.

Canals are artificial Channels construoted for irrigation purposes or for town water supplies or for navigation purposes (in England). When Akbar the Great was Emperor in India at Delhi in 16th Century A.D. he ordered Western Jumna canal to be built on the old alignmcnt of a Channel built in 14th Century by Emperor Ferozeshah Tuglak to carry water to a hunting lodge in Hissar. In 1626, Alimardan, the great Moghul Engineer constructed a branch to irrigate lands in Karnal, Rohtak and Delhi Districts.

In Punjab and in Sindh the rivers rise in floods during the season of June to September and carry large volumes of silt-laden water to the Sea at Karachi.

The ancients constructed inundation canals, so called bccause they carried flood waters only, taking them off from some of these rivers, carried them for miles along water shed lines to irrigate tho fields. The cxamples are canals in the districts of Deragazikhan, Muzzafargarh, Multan and north Sindh.

These canals had the disadvantage of depending for their supply of water on floods in rivers, which were sometimes low and sometimes high.

The Engineers were probably Arabs imported from Egypt, where the Nile river and the canals therefrom were the source of irrigation and ancient civilization.

These inundation canals carry water in wot season only and were built before the British conquered India, some were big enough to be used both for irrigation and navigation purposes.

The widths and depths as well as the bed slopes of these canals were so proportioned that the velocity of moving water at all depths of flow was sufficient to carry along silt and thus prevented silting of the canal; the silt was transported through distributaries to fields where it deposited and acted as a fertilizer medium.

The Engineers knew Kennedy Law by experience. The writer has observed in the caso of canals in upper Sindh that tho depth of the canal varies from one fourth to one third of the bed width in the case of main channel as well as distributaries.

It is a pity that records of the observations of these Engineers are not preserved. They had no modern scientific instruments yet their works are results of scientific thought and principles.

In Burdwan district (Delta of river Damodar), in Tanjore district (Cauvery Delta), canals were built several centuries ago.

The British Government have remodelled all these old canals and provided them with modern Head regulators, bridges and outlets.

India owes a great debt of gratitude to the British Engineers whose names will be cherished into the memory of all those who have benefited by the great Engineering works carried out during the British occupation of India.

Chapters XI and XII deal with principles of design for canals carrying more or less clear water.

109. Canals carrying silt laden water

(*a*) *Movements of solids by a stream.*

When flowing water transports solid substances by carrying them in suspension, they are known as 'silt'; when by rolling them along the channel, they are termed "drift." The weight of silt present in each cubic foot of water is called the charge of silt. Silt means very fine and light particles of clay, mud and sand. Drift means sand, gravel, shingle and boulders.

When a stream obtains materials by eroding its channel, it is said to scour. When it deposits materials in its channel, it is said to silt.

A stream of given velocity and depth can carry only a certain charge of silt. When it is carrying this, it is said to be fully charged. If a stream is not fully charged, it tends to become so by scouring its bed.

If we examine the contents of water and silt deposits at about the Head of a canal and at other different sites on its way to its tail, we find that the contents are different. When the velocity is high enough, it carries coarse silt, when poor, the heavy grades of silt deposit in the upper reaches of canals. As we go down the canal, the proportion of heavy grades of silt becomes less and less and so on till the very fine silt is left at the tail.

Very flat slopes and low velocities are sufficient to carry very fine silt in suspension.

During summer rains, there is more silt and consequent deposit in the canals, creating steaper grade in the water surface levels. In winter, some of the silt is washed down further below and the slope flattens.

A channel is said to be in steady regime when the state of flow is such that the bed and sides remain constant in position and grade. By "final regime" is meant the final steady state of a channel reach, which has formed its own section and slope in its own silt.

It has been observed that both the silt charge and grade increase from the surface to the bed of the channel, also the silt charge varies from day to day, from season to season.

(*b*) *Discovery of Kennedy Law.*

The British Engineers in Punjab designed canals on the basis of Chezy and Kutter's formulæ as given in article No. 88 of this book, taking the value of n as 0·0225 for channels in alluvium. They took no account of the sand, pebbles and silt coming into the canal.

Early in the nineties of last century, silt troubles arose rather in an acute form in the head reach of Sirhind canal, which takes off from the river Sutlej at Rupar. The head reach silted up almost solidly for miles, and lakhs of Rupees were spent on its clearance every year. This was a problem for the Engineers to solve. It was the genius of Mr. R. G. Kennedy, the then Engineer in charge of Bari-Doab canals at Amritsar, which solved this problem.

He made observations on Bari-Doab canal and its distributaries, which he found in steady regime, neither silting nor scouring and evolved his well known formula.

His publications are as under.

Paper No. 2826, session 1894-95 of the proceedings of the Institution of Civil Engineers, London.

Hydraulic Diagrams for design of channels.

Appendix A of the publication No. 20 of the Central Board of Irrigation, Government of India.

His diagrams for designing channels in alluvial soil in Punjab are very valuable. He rightly takes his stand on Chezy formula, $V = C\sqrt{r}\sqrt{s}$, taking Kutter's expression for C as given in Kutter's formula article 88. In this the value of C depends upon 's', the slope of the Channel, 'r', the Hydraulic mean radius and 'n', the coefficient of rugosity, which has been found 0·0225 for channels in Punjab.

Enginsers in the British Empire have come to the conclusion that Chezy's formula and Kutter's expression for the value of 'c' give more reliable and constant results than any formula yet known. Mr. Kennedy explains his theory and formula as under.

He deduced from observations made on the upper Bari-Doab canal that for non-silting and non-scouring channels in steady regime there is always one velocity and that velocity is a function of depth of flow in the channel. He calls this "critical velocity." When water flows along the bed of the channel, eddies are formed which travel vertically upwards, get weaker as they approach the surface and maintain the silt in suspension. Eddies from the sides of the channel travel horizontally and have no silt supporting power; the turbulent flow thus produced keeps the silt in suspension for miles long in the channel.

He calls his critical velocity Vo: We call it Vc. Thus $V_c = 0{\cdot}84\ D^{0.64} \times C$ (silt coefficient) ....80.

He lays down that velocity greater than Vc for a particular depth will scour and velocity less than Vc will deposit silt.

The value of C depends upon the character of the silt. Thus C = 1·00 for fine light sandy silt in the rivers of northern India and in Bari-Doab canal.

= 1·10 for somewhat coarser light sandy silt.

= 1·20 for sandy loamy silt.

= 1·30 for rather coarser silt or debris of hard soils.

= 0·70 for silt of River Indus in Sindh.

As regards the quantity of silt in suspension, Mr. Kennedy found that if $C = 0 \cdot 90$, only 77 per cent. of the full charge of Northern India silt can be carried along and if $C = 0 \cdot 80$, about 57 per cent.

The depth of non-silting and non-scouring flow is often required to calculate the depth of foundations for abutments and piers of bridges.

For this purpose the following formula of Kennedy is useful.

$$D = \frac{(Vc)^{1.56}}{m} \quad \ldots\ldots 81.$$

where D = depth of non-scouring and non-silting flow.

Vc = critical velocity which causes neither silting, nor scouring

m = scour coefficient for the material of which the bed is composed.

For gravel and boulders m varies from $\cdot 8$ to $3 \cdot 5$: M.E.S. Hand book Volume III, 1935 Edition, pages 38 to 48 suggests that foundation should be taken to a depth of $1\frac{1}{3}$D to allow for factor of safety. This gives reasonable results.

The erosive power of water varies as the square of velocity, transporting power varies approximately as V^6. The size of a particle moved by a stream over a smooth sandy bed is given approximately by the formula, $d = \frac{45\ V^2}{W - 64}$ inches ......82.

where W is the density in lbs. per cubic foot of silt and V is the velocity in feet per second ('Engineer' May 15, 1908, page 511).

If V_1 be the velocity to carry Q_1 the quantity of silt, and V_2 be the velocity to carry Q_2, different quantity of silt, then $Q_2 = Q_1 \left(\frac{V_2}{V_1}\right)^{\frac{6}{5}}$.

The full depth of flow is 10 ft., generally $\frac{1}{3}$ to $\frac{1}{4}$ of bed width in SINDH Inundation canals, dug through firm loamy soil by Muslim Engineers in ancient times, who maintained $3 \cdot 5$ ft. per second as non-silting and non-scouring velocity, for the silt carried during floods.

Mr. Kennedy prepared a set of diagrams to design channels in Punjab so as to prevent silting and scouring in canals. His critical velocity is a function of the depth of flow and he omits the width of the channel or the value of r, the hydraulic mean radius, in his formula. Some Engineers in Punjab made use of his diagrams improperly with the result that channels built by them silted very badly. Many channels designed from his diagrams have worked well and absence of silt deposit in them has saved Government large sums of money.

The draw back of Kennedy's theory is that it allows channels to be designed in an infinite number of ways without any consideration of what the channels were trying to settle down to permanent section (Blench).

To make Kennedy's diagrams more useful, Mr. Woods prepared his tables of normal data of design of Kennedy's channel, which give ratios of bed to depth for various discharges, this enabled the Engineers to design bed widths and depths rightly (Punjab).

Mr. Lindlay, also an eminent Engineer in Punjab P.W.D. seeing that some channels designed from Kennedy's diagrams failed badly, made experiments on lower Chenab Canal and presented his results before the Punjab Engineering Congress in 1919. He says that "Nature ordained only certain bed widths (neither more or less) for particular discharges in silt laden water. This means that there is a sort of relationship between the discharge and dimensions of channel."

This statement of Mr. Lindlay is supported by the fact that River Indus in North Sindh, where it flows along the watershed line, the country sloping away from it, has formed its own channel in the alluvial soil brought down by the river as silt during floods.

TABLE No. X.

Showing critical velocities for various Depths (Garrett).

$V_c = C \cdot 84 D^{\cdot 64}$, see formula 80.

Depth in feet D	Critical velocities corresponding to values of C, the silt coefficient.					
	·8	·9	1·0*	1·1	1·2	1·3
2	1·04	1·17	1·30	1·43	1·56	1·69
2·5	1·21	1·36	1·51	1·66	1·81	1·96
3	1·36	1·53	1·70	1·87	2·04	2·21
3·5	1·50	1·69	1·88	2·07	2·26	2·34
4	1·63	1·84	2·04	2·24	2·45	2·65
4·5	1·76	1·98	2·20	2·42	2·64	2·86
5	1·88	2·11	2·35	2·59	2·82	3·05
5·5	2·00	2·25	2·50	2·75	3·00	3·25
6	2·11	2·38	2·64	2·90	3·17	3·43
7	2·34	2·63	2·92	3·21	3·50	3·80
8	2·54	2·86	3·18	3·50	3·82	4·13
9	2·74	3·09	3·43	3·77	4·12	4·46
10	2·93	3·30	3·67	4·04	4·40	4·77
11	3·12	3·54	3·90	4·29	4·68	5·07
12	3·30	3·71	4·12	4·53	4·94	5·36

* Applicable to Bari-Doab Canal of Kennedy.

Velocities transporting different materials.

Material.	Bottom Velocity in feet/sec. at which		
	Transportation begins.	Material is in equilibrium.	Deposit begins.
Coarse sand	1·07	·71	·62
Gravel, size of pea	·71	·62	·71
Gravel, size of small bean	1·56	1·07	·71
Shingle, rounded one inch or more	3·20	2·14	1·56
Flints—size of hen's egg	4·0	3·2	2·14

(c) Mr. K. R. Sharma, also an eminent Engineer in P.W.D., Punjab presented his paper "Design of Irrigation Channels" to the Institution of Engineers (India) in 1936 and has gone thoroughly into the question of the design of canals carrying silt laden water. He supports Kennedy's theory and suggests some useful improvement in his paper, which is worth reading.

According to Kennedy, the amount of silt per one Cubic foot of water varies as

$$\frac{V^{\frac{3}{2}}}{d\ (\text{depth of flow})} \qquad \ldots\ldots 83.$$

Effect of outlets on the grade of silt.

Let d represent the diameter of the average particle of silt in a channel section, then

d^2 Q varies b V^{3n} ......84.

the value of n being $1\frac{1}{2}$ in the case of Punjab canals.

TABLE No. XI.

Multipliers for reducing velocities and discharges in trapezoidal channels calculated for side slopes of $\frac{1}{2}$ to 1 (see Table VI) to corresponding velocities and discharges with side slopes, vertical, 1 to 1, and $1\frac{1}{2}$ to 1 :

Bed width / depth	Multipliers for Velocity			Multipliers for Discharge		
	Vertical	1 to 1	$1\frac{1}{2}$ to 1	Vertical	1 to 1	$1\frac{1}{2}$ to 1
·8	..	1·12	1·17	..	1·55	2·05
1·0	·79	1·09	1·13	·52	1·46	1·88
1·2	·82	1·07	1·09	·57	1·39	1·75
1·4	·84	1·06	1·08	·62	1·34	1·64
1·6	·86	1·05	1·06	·66	1·30	1·56
1·8	·87	1·04	1·05	·69	1·27	1·50
2·0	·887	1·036	1·040	·711	1·245	1·458
2·5	·913	1·023	1·022	·760	1·194	1·361
3	·927	1·015	1·010	·795	1·160	1·300
3·5	·939	1·011	1·005	·822	1·137	1·257
4	·947	1·008	1·001	·841	1·120	1·224
4·5	·953	1·007	1·000	·857	1·108	1·198
5	·956	1·004	·997	·870	1·096	1·178
6	·964	1·003	·995	·890	1·080	1·148
7	·970	1·001	·993	·905	1·068	1·126
8	·974		·993	·916	1·059	1·110
9	·978		·993	·926	1·053	1·097
10	·980		·992	·935	1·048	1·087
12	·985		·993	·946	1·039	1·072
14	·988		·993	·954	1·033	1·061
16	·990		·993	·960	1·029	1·053
18	·991	1·000	·994	·964	1·026	1·047
20	·992		·994	·968	1·023	1·042
30	·995		·995	·979	1·015	1·028
40	·997		·996	·984	1·012	1·021
50	·997		·997	·987	1·009	1·017
70	·998		·998	·991	1·007	1·012
100	·999		·998	·994	1·005	1·008

How to design section of the channel :—

Usually the slope is determined from the lie of the country, and the quantity of discharge Q is also given. From the nature of the soil, the value of N is also determined. Over thirty years experience of flow in the irrigation canals of India has led Engineers to the adoption of the following values of N for the design of new channels. $N = \cdot 02$ for new canals carrying over 400 cusecs and $N = \cdot 0225$ for all smaller channels. As weeds and grass grow on the sides of the channel sooner or later, the value of N as $\cdot 0225$ is adopted.

Ex. I :—It is proposed to construct a canal through ordinary firm loamy soil in which the canal section will stand side slopes of ½ to 1. The river, from which the canal is to take off, is snow-fed and carries *coarser light sandy silt*, a good amount in rains and less amount in dry season. The coefficient c of critical velocity Vo is assumed as 1·10 : The required discharge is 5000 cusecs. The available fall of the country is 1–5000. The maximum velocity which the soil can stand without erosion is 3·5 feet per second ; work out the cross section according to Kennedy's Theory and Kutter's formula, the coefficient of rugosity N being ·0225 ?

Send a quantity of river water fully charged with silt to nearest Hydraulic research laboratory and ascertain the maximum diameter of silt particle. It is always necessary that silt ejectors should be installed at the head-works of the canal to exclude stone, heavy silt, and heavy sand.

The non-scouring and non-silting depth of flow in the canal is given by the Formula No. 80 : $Vc = 0 \cdot 84 D^{\cdot 64} \times C$. The value of C in our case is 1·10 : and Vc is 3·5 ft./sec. maximum. Then D is 8 ft. from Table X.

From Nomogram No. 6, Fig. 77, with s as 1/5000, V as 3·5 ft./sec., and N as ·0225, value of R, the Hydraulic mean Radius is about 8. As the discharge is 5000 cusecs, the area of water-way is $\frac{5000}{3 \cdot 5} = 1428$ sq. ft. : wetted perimeter is $\frac{1428}{8} = 178 \cdot 5$ feet. The depth being 8 feet, side slopes ½ to 1, the bed width comes to 174·5 feet. Thus the canal is to be 174·5 feet wide at bottom, depth of flow 8 ft. : Bed slope 1-5000, Q = 5000 c. secs. mean velocity 3·5 ft./sec.

From Nomogram No. 5, Fig. 76, $c = 90 : R = 8 : N = \cdot 0225 : \sqrt{S} = \cdot 0141 : V = c\sqrt{R}\sqrt{S} = 3 \cdot 5$ ft./sec.

If the level of water in the canal falls, the velocity decreases and transport of silt is affected. See Table X.

Suppose now, our canal is required to carry 2500 cusecs. only. To carry on the silt, Velocity must remain 3·5 ft./sec. In that case the area of water section is $\frac{2500}{3 \cdot 5} = 714 \cdot 3$ sq. ft. To maintain the critical velocity, the depth of flow must be 8 ft. and the H. M. R as 8 feet. The wetted perimeter is now $\frac{714 \cdot 3}{8} = 89 \cdot 3$ ft. The side slopes are ½ to 1 : our water section now is : bed width 71′–3″ : depth 8 feet. Bed slope 1–5000 :

Thus a non-silting and non-scouring channel can be easily designed to suit any bed slope. It is the depth of flow that controls the transport of silt. Kennedy's Diagrams, modified by eminent engineers are nowadays available in the market.

Kennedy based his theory on the charge and grade of silt present in Bari-Doab canal during the last quarter of 19th century. He fixed the value of c in the formula No. 8 as unity for Bari-Doab canal silt, which is now of different grade as reported by engineers in charge : Thus the formula No. 80 depends upon the nature and quantity of silt present in the parent channel from which the canal takes off.

(*d*) Mr. Gerald Lacey, another eminent Engineer of P.W.D. in United Provinces of Agra and Oudh in India, finding that Mr. Kennedy did not take cognizance of bed width and Hydraulic mean depth of a channel in his theory, brought out after great thought and labour his following theory.

The velocity, the hydraulic mean depth, the wetted perimeter, the rugosity of channel sides, the quantity of discharge, the bed slope had all among themselves a correlationship to produce flow in a canal which will prevent silt deposit, scouring of the bed and sides and growth of weeds, in other words to keep the channel in first class " regime ". (The ancient Engineers had some knowledge of this theory as proved by their silt canals in Sindh and Punjab.)

His contributions to the science of Hydraulics are unique and his publications are:—

(*i*) Paper No. 4893: Institution of Civil Engineers (London) also Vol. 229 and Vol. 237; (*ii*) U.P. Technical Paper No. I; (*iii*) Paper No. 4736 I.C.E. (London); (*iv*) Government of India, Central Board of Irrigation. Paper No. 20; (*v*) Hydraulic Diagrams.

Lacey's formulæ.

V_c = actual Regime velocity in a canal called critical velocity.

$V_c = 1{\cdot}17\ \sqrt{f}\ \sqrt{r}$ .... 86.

f = Lacey's silt factor, varying from 0·4 to 1·60 for Punjab Canals.

= 0·62 for Rohree canals taking off from the river Indus where the silt is fine.

= 0·9 for Ganges at Sara.

= 0·6 to 0·7 on Sarda canal.

= 2·2 to 2·8 generally: greater figures for greater rugosity.

= 1 on Bari-Doab canal. Here $V_o = V_o$ of Kennedy.

= 6 for streams, carrying boulders.

= 4 for hill streams carrying shingle.

The value of f depends upon rugosity of channel, silt-grade.

d = Diameter of silt in a canal or river in inches.

$= f^2/64$ .... 87.

A rough qualitative formula for the diameter of silt in inches, for the predominant type of silt transported. (I.C.E. London Paper No. 4736).

This is very useful for a new canal project.

$f = 8\, d^{\frac{1}{2}}$ (Punjab Research Institute) .... 88.

n = Kutter's coefficient of rugosity: = 0·018 to 0·025 for Punjab canals: = 0·0225 average for Punjab canals: $= 0{\cdot}022\, f^{\frac{1}{4}}$: f = 0·41 to 1·51 if n varies from ·018 to ·025.

A = area of the canal or channel.

P_W = wetted perimeter of the channel.

Q = Discharge in cubic feet per second.

s = Bed slope of the channel. say 1 in 5000 or $\frac{1}{5000}$.

$Af^2 = 3{\cdot}8\, V_c^5$ .... 89.

$P_W = 2{\cdot}668\, (Q)^{\frac{1}{2}}$ .... 90.

2·668 is a coefficient varying from 2·20 to 3·20 depending upon the nature of the soil: for canals, in alluvium soil, it is 2·668.

$Q\, f^2 = 3{\cdot}8\, V_c^6$ .... 91.

$V_c = 16{\cdot}05\, (r^{\frac{2}{3}})\, (s^{\frac{1}{3}})$ .... 92.

$= \frac{1{\cdot}3458}{n}\, (r^{\frac{3}{4}})\, (s^{\frac{1}{2}})$ .... 93.

$r = 0{\cdot}472 \left(\frac{Q}{f}\right)^{\frac{1}{3}}$ .... 94.

$$s = \frac{f^{\frac{5}{3}}}{1844 \cdot 3\, Q^{\frac{1}{6}}} \qquad \ldots\ldots 95.$$

$$= 0 \cdot 391 \quad f^{\frac{5}{3}} \times Q^{\frac{1}{3}} \qquad \ldots\ldots 95A \text{ (Blench).}$$

$$= \frac{V}{r} \quad \frac{(Vs)^{\frac{1}{2}}}{64 \cdot 2} \qquad \ldots\ldots 96. \text{ (Dr. Malhotra).}$$

s = Regime slope.

$$f = \frac{(v\,s\,0/00)^{\frac{1}{2}}}{\cdot 4478} \qquad \ldots\ldots 96A. \text{ (G. L. Bhandari)}$$

formula No. 95 is mathematically sound because the slope automatically averaged out the effect of all the varied sections in the reach over which slope was measured.

Slops measured from water surface takes into account the slight element of shock, differentiating a working slope from a regime slope and thus produced a working silt factor of great value. (Lacey).

The above formula have been worked out by Lacey from statistics relating to the actual flow of water in canals in Punjab and United Provinces of Agra and Oudh (India).

Lacey has taken into account the modern energy theory of flow of water in channels.

His formulæ have received general support from Engineers all over the world.

Steady streams flowing with constant slope and discharge have related wetted perimeter and sectional area, change in one will naturally involve change in the other.

Lacey's formula No. 91 above gives figures much in excess of actual bed widths of ordinary hill streams. In the case of major streams in hills with discharge of 200,000 cusecs, it gives sound results.

The depth upto which scour takes place near the foundation of abutments and piers of bridges is given by the following formulæ of Lacey.

$$D = 0 \cdot 9 \left(\frac{q^2}{f}\right)^{\frac{1}{3}} \qquad \ldots\ldots 96B.$$

D = depth of non-scouring flow below high flood level.

q = average intensity of flood per foot run of water way

$= \frac{Q}{W}$: Q = full discharge : W = full width.

The actual foundations of abutments should be taken about 4 feet below D.

Lacey's theory has been accepted by the Central Board of Irrigation, Government of India (July 1934) and his diagrams are also a standard book in the irrigation department of United Provinces of Agra and Oudh in India.

The silt factor "f" given in the formulæ is an enigma to most engineers. Mr. Lacey and his supporters, Mr. Montague and others explain it as under.

In nature there is no such thing as absolute and long continuing regime; canals flow in regime for years and then silt up due to slow change in the grade and charge of silt; for example the upper Bari Doab canal, including the tails, are deteriorating as a result of silt influx now a days.

Lacey's theory postulates a constant discharge flowing in an envelope made of incoherent, self borne alluvium and carrying a constant silt charge. These conditions never exist in practice in either natural or artificial channels, except as an accident and instantaneously; therefore allowances are made, taking into consideration the practical conditions.

The silt factor "f" is a factor which takes into account, the actual volume of the silt charge, its density, its shape and indeed any other attribute of silt. The engineer must intelligently estimate the value of 'f', based on such data as available, in other words, experience. "f" is a true physical constant of the silt content and not like V_0 (Kennedy's C. V.) which varies with discharge.

The slope of the channel is determined along with all other dimensions by the discharge and silt factor.

Lacey's silt velocity formula connects the silt transported at a given velocity with the hydraulic mean depth and primarily depends upon the silt charge transported.

The silt factor "f" represented the physical effects on the channel and its shape caused by the silt burden, and the value of "f" and tho volume of discharge uniquely determined the slope and all other hydraulic dimensions.

No doubt it is very difficult to estimate the value of "f" which would obtain in a new channel after new construction. But it had to be done and here the capacity and experience of the designer is his degree of success in the very process.

Channel shape in incoherent alluvium is, in fact, essentially a balance between bed, sand, and fine silt in motion; also on the mean diameter of the particle of silt and also on relative quantities of fine and coarse particles in movement.

The hydraulic mean depth also is affected by silt grade. "f" characterises the bed silt grade also.

For constant silt grades, the ratio of bed width to depth or more accurately that of the wetted perimeter to the hydraulic mean depth, steadily diminishes with reduction in discharge.

The shape of the channel $\frac{P}{r}$ for a given discharge is a function of the silt grade; channel in finer material being narrower and deeper.

For a given grade of silt, the regime slope required will increase with reduction in discharge.

If a main canal terminates into two branches, the regime slope of the branches must be greater than that of the main canal.

Regime slope is determined by the grade of silt admitted. If the regime slope is less than what is available, falls, as necessary are interpolated. If tho available slope is insufficient, attempts at reducing the grade of the silt must be made by improving regulation in the case of a canal head or the distributary.

"f" is purely a silt factor and an individual canal owes its shape to size of silt on its bed and roughness of its sides and a uniform "f" cannot be fixed for a wholo system of canals for design. For Haveli canals, 'f' was taken as 0·8 throughout. This was a mistake becauso the soil was not the same for the whole system.

Only oxperts could decide what value of "f" might be adopted for designing new channels or remodelling old ones.

"f" is a characteristic which related to silt grade, to discharge and slope in a regime channel.

The physical meaning of "f" is not known but it is a physical constant of "silt content" and appears to depend mainly on the diameter of the most prevalent silt below 0·6 millimeter diameter in channels carrying silt only (and not boulders).

Mr. Lacey says that silt factor "f" is proportional to $\frac{V^2}{r}$ which is proportional to $\frac{V^2}{gr}$, the dimension less Froude number and a criterion of dynamic similarity.

The ratio $\frac{V^2}{r}$ is a criterion of regime and also it is a function of silt grade; for that reason it is called silt factor. With increase in the numerical value of the silt factor, fino silt would be replaced by coarso silt, coarse sand by shingle, and shingle by boulders. "f" is a physical quantity that defines regime channel conditions and Lacey's formula defines "f" in terms of functions that we can measure.

The product r × v was clearly the hydraulic discharge per foot run denoted by 'q'; in all regime channels $\frac{q}{P}$ is constant.

See paper No. 187 "Scientific irrigation channel design" by *T. Blench,* Punjab Engineering Congress 1936.

In Sindh the silt problem is complex due to the coherent nature of silt, resulting from large percentage of colloidal matter. f = 0·62 for Rohrco canals, because the silt is very fine.

Paper No. 233 P. E. Congress (1940) on "Shock in Regime Channels" by Gerald Lacey is worth reading. It shows the high order of Mathematical genius of Mr. Lacey. This theory is under test and may be adopted by the Government of India at a later date.

No great abnormality of design is required to ensure 'f' being constant over a distance of say fifty miles.

Proportional silt distributors divide silt content equally.

Regime of a canal can be tested for by working out 'f' by the slope formula for several channols of an area and noting those whose 'fs' agree.

Lacey's theory does not apply to Bombay Deccan canals because Deccan silt is Non-coherent; see "Economics of Deccan canals: selection of water depth": journal of the Institution of Engineers (India) Vol. XX, January 1941.

Lacey's theory requires certain ratio of water-width to depth for regime channels, having velocities shown below.

TABLE No. XII.

Velocity	Ratio : surface width/ water depth
0·88	2·0
1·00	3·6
1·50	7·1
2·00	10·1
2·50	13·1
3·00	16·0
4·00	21·8

Silt load, parts by weight per 100000 of canal water in Jamrao canal in Sindh : (*a*) average for the year 100 : (*b*) Maximum 400 : size of silt particles in suspension 0·02 to 0·20 (millimetres).

Example 2 :—Now examine the example 1 in the light of Lacey's theory : see formula No. 80. The silt coefficient of Bari-Doab canal silt is 1·00 (Kennedy). Lacey's silt factor 'f' for Bari-Doab canal is also 1·0, same as that of Kennedy. In example No. 1, the silt is coarser than that of Bari-Doab canal and therefore the value of C in formula 80 has been taken as 1·10 : Lacey's silt factor 'f' should also be taken as 1·1 at least. An experienced Engineer should be consulted about it.

In formula 86, $V_0 = 1 \cdot 17 \sqrt{f}\sqrt{R} = 1 \cdot 17 \times \sqrt{1 \cdot 1} \times \sqrt{8} = 3 \cdot 47$ against 3·5 f/s in example No. 1. In formula 87, d, the diameter of silt $= \frac{f^2}{64} = \cdot 019$ inches. In formula 88, $f = 8d^{\frac{1}{2}}$: d = ·0189″. Silt ejectors should be constructed to keep out silt particles over 0·019″ in diameter or even less. In formula 89, $A f^2 = 3 \cdot 8 V_c^5$: $A \times 1 \cdot 1^2 = 3 \cdot 8 \times 3 \cdot 5^5$: A = 1646 □′ : against our figure 1428 □′ : In formula 90, $P_W = 2 \cdot 668 Q^{\frac{1}{2}} = 188 \cdot 65$ feet against our figure 178·5 feet. In formula 91, $Qf^2 = 3 \cdot 8 V_c^6$: Q = 6970 against our figure 5000 cusecs.

Also see Table No. XIII : Read the line, against the discharge Q 5000 cusecs : The figures for the wetted perimeter, Hydraulic mean depth, and velocity agree approximately with the figures in example No. 1 where C is taken as 1·1 against f as 1·0 in Table No. XIII.

(*e*) Formulæ of Punjab Research Institute. In Paper No. 252, "Designs of Channels in Alluvium" submitted before the Punjab Engineering Congress in 1942, Dr. N. K. Bose and Mr. K. R. Erry, Officers of the Irrigation Research Institute, Lahore, gave the following formulæ :—

$$s \times 10^3 = 2 \cdot 09 \frac{m^{\cdot 85}}{Q^{\cdot 21}} \qquad \text{.....96C.}$$

Where m = diameter of silt particle on bed in millimetres.

s = slope of the channel.

Q = Discharge in cusecs.

Mr. Bose actually experimented upon silt with diameter 0·6 mm. (·023″).

$$A \text{ (area of waterway)} = 1 \cdot 145\, Q^{\cdot 85} \qquad \text{.....97.}$$

$$D \text{ (depth of flow)} = 0 \cdot 39 \frac{Q^{\cdot 29}}{S^{\cdot 37}} \qquad \text{.....98.}$$

Here S = slope per thousand feet.

$$D \text{ (depth of flow)} = \frac{0 \cdot 297\, Q^{\cdot 368}}{m^{\cdot 314}} \qquad \text{.....98A.}$$

(C. C. Inglis of Sindh).

Dr. Bose

$$P \text{ (wetted perimeter)} = 2 \cdot 800\ Q^{\frac{1}{2}}$$

$$r \text{ (Hydraulic mean radius)} = 0 \cdot 47\ Q^{\frac{1}{3}}$$

$$v \text{ (velocity in feet per second)} = 1 \cdot 12\ r^{\frac{1}{2}}$$

$$w = \text{width of water surface} = 3 \cdot 857\ Q^{\cdot 482} \times m^{\cdot 318} \qquad \text{......................98B.}$$

The above formulæ were derived after exhaustive observations on lower Chenab canal system at various sites in regime and are stability formulæ for typical stable channels in Punjab.

Equation No. 98 gives the value of D and equation 97 gives the value of A, the area of the waterway. Then the width of the water surface is found, supposing the cross-section to be rectangular.

Fig. 98.

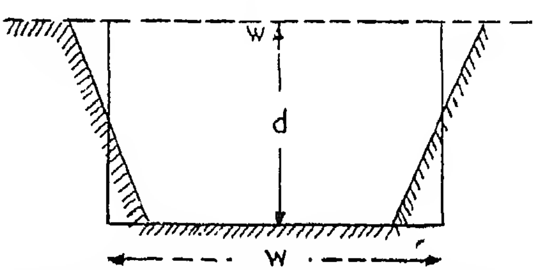


Fig. No. 98 explains this fully. $A = w \times d$; the waterway is a rectangle and from the middle points of the sides, draw the side slopes $\frac{1}{2}$ to 1 or 1 to 1 or $1\frac{1}{2}$ to 1 according to the nature of the soil. The nature of the berm soil determined the width and consequently the wetted perimeter of the channel.

The above formulæ can only be used for designing channels in clayey and slightly sandy soil similar to that of Chenab system in Punjab.

Three Nomograms are given No. 7 (Fig. 100C) : No. 8 (Fig. 100D) : No. 9 (Fig. 100E).

It may be noted that in equation No. 96, the value of 'm' was obtained by analysis of the particles of bed silt whose maximum size was 0·6 mm. This formula would need modification for coarser or finer sand than given above. The formula 97 would not apply to canals in Sindh or of Sutlej valley project whose silt factor is very low, the silt being finer.

The slope is a vital factor in regime design and it is necessary to know if the required slope was available.

The independent variables of a channel in fine alluvium are slope and silt 'charge and grade'; the soil characteristic is a third variable, and this determined the ultimate dimensions of a channel, which is called a regime channel if it be in stable equilibrium under the conditions of discharge, silt charge and grade and bank conditions when tested for a very long time.

(f) L. Ishwardas' observations and slope formulæ. In paper No. 249 (Punjab Engineering Congress 1941), L. Ishwardas, an able Engineer of Punjab Irrigation Department discusses fully "The Hydraulics of Irrigation Channels."

He has plotted improvements on Kennedy's diagrams and has shown that Lacey's 'f', the silt factor, is an enigma. He has also prepared a slide rule for solving problems in designs quickly; he has prepared various statements from the data and observations on various canals and these are given in proceedings of the Punjab Engineering Congress (1941).

His talented daughter Miss Shanta, B.A. has brought out a slope formula which is given below.

Miss Shanta's formulæ.

$$s^{\frac{10}{3}} = \cdot 0167\ Q^{\frac{3}{2}} \qquad \text{..98C.}$$

$$s^{\frac{10}{3}} = \frac{n^2}{12 \cdot 1844}\ \frac{Q}{r^{\frac{1}{2}}} \qquad \text{..98D.}$$

n = coefficient of rugosity.

TABLE No. XIII

(Hydraulics of irrigation channels, P.E.C., (1941).

Discharges, Wetted Perimeter, Hydraulic Mean-depths and Velocity. (Lacey's Formulæ).

$f = 1$ Ishwardas.

Q	$Q^{\frac{1}{2}}$	$Q^{\frac{1}{3}}$	$P = 2\cdot67\ Q^{\frac{1}{2}}$: $R = \cdot48\ Q^{\frac{1}{3}}$: $V = 1\cdot13\sqrt{R}$ $P \times R \times V = Q$ P	R	V	Q
10	3·16	2·154	8·44	1·03	1·15	10·00
50	7·07	3·68	18·88	1·77	1·50	50·13
100	10·00	4 642	26·7	2·23	1·69	100·62
150	12·242	5·313	32·7	2·55	1·80	150·09
250	15·81	6·299	42·21	3·02	1·96	249·8
500	22·36	7·937	59·7	3·81	2·81	502·7
1,000	31·6	10·0	84·4	4·8	2·48	1,004·7
2,000	44·72	12·60	119·04	6·05	2·78	2,008·0
5,000	70·71	17·10	188·62	8·20	3·24	5,008
10,000	100·0	21·54	267·0	10·34	3·63	10,022
20,000	141·42	27·14	377·6	13·03	9·08	20,074

P = Wetted Perimeter.

R = Hydraulic Mean-depth.

V = Velocity, feet per second.

Q = Discharge in cusecs.

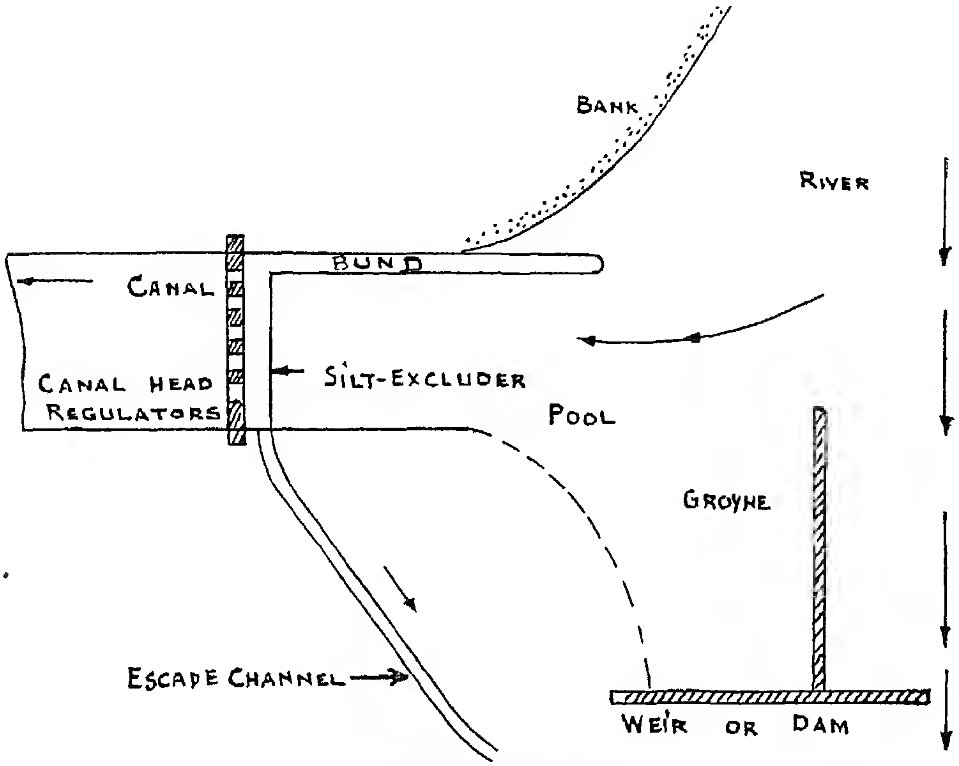
Example No. 3 :—Now examine the example No. 1 in the light of formulæ of Punjab Hydraulic Research Laboratory at Lahore. Our diameter of silt is ·0189 inches or ·5 millimetres : slope 1–5000 or ·2 in 1000 ft. Discharge is 5000 cusecs. In nomogram No. 8, Fig. 100D a straight line passes through our above quantities. They all tally. From nomogram No. 9, Fig. No. 100E, area of waterway is 1700 □' against 1646 in example No. 2 and depth of flow is 8 feet, same as our figure in example No. 1 and example No. 2.

For designing canals and distributaries, it is necessary to have Kennedy's tables and diagrams (Improved), or Mr. Lacey's diagrams. In the absence of above books, the tables prepared by Mr. Ishwardas, the nomograms Nos. 7–8–9 prepared by Dr. Bose, will be of great help in solving the problems. The nomogram Nos. 5 and 6 based on Kutter's formula, will solve the problem of designing a channel to carry clear water, free from silt. To make an allowance for silt, equation No, 80 will help to give the critical velocity. Thus by following the data given for existing canals, various problems can be solved satisfactorily.

The names of the eminent engineers, mentioned in this Chapter, who have worked so hard on the solution of this difficult subject, will ever be cherished in our memory.

(*g*) Silt excluders :—

Fig. 99



We have seen that coarse silt, which deposits in canals, is a source of great trouble to canal Engineers, who have found it necessary to exclude it at the canal head regulators. Silt excluders have been devised and built at the head of several canals in Punjab with success. Fig. No. 99 shows an arrangement of that kind. Coarse silt settles in the pool created in front of the regulator and a silt excluder was also built in front of the regulator; the excluder is a mechanical separator of the upper comparatively silt free water from the lower water with greater concentration of silt. This mechanical separation postulates that the approaching stream was flowing in stream line flow or Laminar flow, and more nearly this condition was fulfilled, the greater the efficiency of the excluder.

The water charged with heavy silt escapes into a separate channel and thence into the river. It is important that water approaching the silt excluder should assume the stream line flow as contrasted with turbulent flow. Stream line flow or flow with Uniform velocity in a straight line might be compared to a state of rest, leading to precipitation of silt to the lowest layers. This is the function of silt excluders.

The approach channel to the silt excluder must have smooth bed to prevent the formation of eddies which produce turbulence.

Sometimes silt extractors are built along the canal to extract silt and lighten the load of silt in the canal.

On this subject, reference may be made to an excellent paper No. 211 "silt excluders" by F. F. Haigh, I.S.E.: read before the P.E. Congress in the year 1938 :

For silting tanks on the Western Jumna canal see paper No. 277 of P.E.C. volume 34 of 1946. These are rectangular tanks, built alongside the canal. Water enters at one end of the tank and gets at the other end. The velocity of flow in the tank is so slow that the silt deposits in the tank.

For canal outlets see paper No. 264 P.E.C. volume 32 (1944).

For flow on curves in open channels see paper No. 242 by A. N. Wilson, P.E.C. volume 29 of 1941.

(*h*) Irrigation outlets :

An outlet is a device at the head of a water course, which connects it with a distributing channel. In America it is termed a " turn-out". The internal working on a water course is managed generally by the cultivators. This device, therefore, must be such that it not only passes a known and constant quantity of water, but must essentially be a measure of the discharge.

Different types of outlets are used in different countries. The number of outlets of different types on the canals in Punjab alone is very large, irrigating about 14 million acres of land. The irrigator is always anxious to have an ample supply of water whenever needed by his crops ; this depends upon the proper distribution of water by suitable outlets.

There are three classes of outlets.

(*i*) Modular outlets or modules whose discharge is independent of the water levels in the distributary and the water course within reasonable working limits.

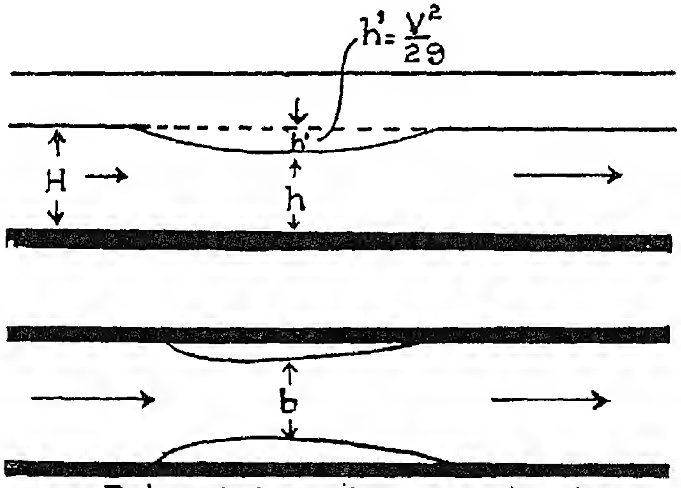
(*ii*) Semi-modules whose discharge although depending on the water levels in the parent channel is independent of the water level in the water course, so long as the minimum working head required for the semi-module is available.

(*iii*) Non-modular outlets, whose discharge is a function of the difference in levels between the water surface in the distributing channel and the water course, variations in either affect the discharge.

For further interesting details on outlets, see the valuable paper No. 264 of P.E. Congress, written by Khan Bahadur S. I. Mahbub, volume 32 (1944). This paper will repay its perusal ; also see paper No. 218 of P.E.C. (1939) for design of an adjustable proportional Module by Mr. N. D. Gupta, I.S.E., also paper No. 237, P.E.C. (1940). Paper No. 128, P.E.C. (1929). Paper No. 125 of P.E.C., 1929, also report of Central Irrigation and Hydro-Dynamic Research Station at Khadakvasla, Poona.

Fig. 99 A

VENTURI FLUME



Rate of Flow = Area × Velocity

$$= hb \times \sqrt{2gh'}$$

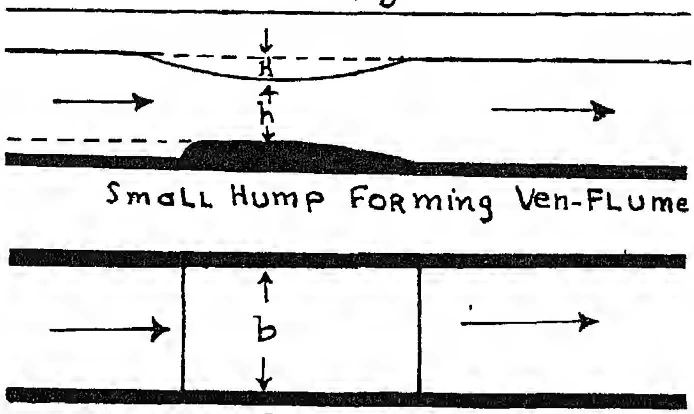
$$= hb \times \sqrt{2g(H-h)}$$

Fig. 99A and Fig. 99B show designs of venturi flumes (" flumen " Latin word meaning river), constructed on open small channels to measure the flow of clear water or sewage, which is recorded automatically by means of floats and charts on drums, revolved by means of clockwork. The arrangement consists of a constriction (in the bed in Fig. 99A and on sides in Fig. 99B) which is stream lined so that the water remains in contact throughout its passage. The dimensions of the constriction and its shape and form are very important. The water surface falls and there is a dip in the water surface due to decrease in the water-way and increase in the velocity. M/s. The Loa Recorder Co. Ltd. of Manchester has perfected recording instruments with floats for sale in the market.

Fig. 99 B.

VENTURI FLUME.

$$h^1 = \frac{V^2}{2g}$$



Rate of Flow = Area X Velocity

$= hb \times \sqrt{2gh'}$

$= hb \times \sqrt{2g(H-h)} \times$ Constant.

To regulate the discharge from rapid gravity filters, also for controlling the rate of flow in pipes, Venturi rate controllers with complicated mechanism have been brought in the market recently by British Firms.

(*i*) Percolation and Lining of canals :—

See standard books on Irrigation practice. Percolation depends upon the nature of the soil, the length of the wetted perimeter, etc., for lining of canals, see paper No. 260 by Mahbub P.E.C. (1943). Canals in sandy soil are lined with concrete or bricks to prevent loss of water by percolation.

110. *Waste Weirs* :—A weir is a masonry wall built across a river to raise its water-level for irrigation purposes. This is sometimes called a pick-up weir.

Fig. 100

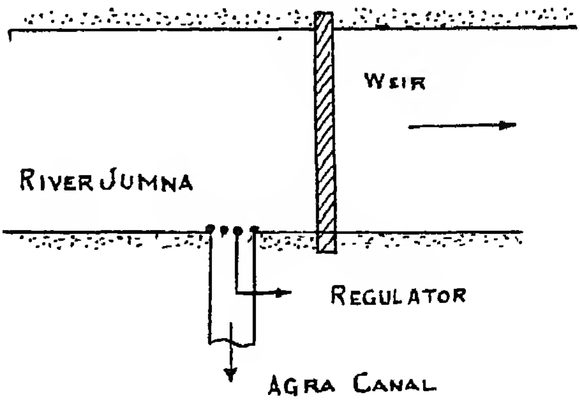


Fig. No. 100 shows plan of weir built across river Jumna at Okla near Delhi. It is built on rock foundations from one side of the bank to the other side. On the right bank is the regulator for Agra Canal. During rains, the flood waters pass over the weir and flow down the river. During dry season, woodon planks (flash Boards) are put up temporarily to regulato the water-surface level in the river for purpose of drawing the supply into the canal.

There is a weir at Srinagar in Kashmir, built across the river Jhelum. Here the water level is raised to create sufficient depth of the water in the river for purposes of navigation. There is also a lock on the left bank at the end of the weir to transfer boats from one side of the weir to the other side. The water passes over the weir during all seasons.

At Khadakwasla near Poona, there is a huge masonry dam across the head of a river, to store water during summer rains in an artificial lake for irrigation purposes. There is also a waste weir at the end of the dam to pass off surplus flood water. This weir is fitted with automatic gates to control and regulate the flood discharge. It is worth-while to visit these works.

The usual formula for discharge over a broad crested weir with free fall is $Q = 3 \cdot 1 \text{ L H}^{\frac{3}{2}}$, where L is the length of the weir, equal to the full width of the channel; and H = head (depth of flow over the weir, measured a little away from the weir + ha the head due to velocity of approach $\left(\frac{V^2}{2g}\right)$. If the length L is unity, $\text{Log } Q = \log 3 \cdot 1 + \frac{3}{2} \log H$. This is a straight line law. Several waste weirs have been built at the end of impounding dams in South India, constructed to store water in artificial lakes or Storage Reservoirs. In such cases, the coefficient in the above weir discharge formula was taken somewhere near 3·0. The stored water was calculated in the artificial lakes on the basis of approximate formulæ of run-off of rain water from the catchment areas. Now it has been ascertained that the length of such weirs in some cases requires to be increased or the increase in the depth of flow over the crest of the weir is to be tolerated, due to unexpected heavy run-off from the catchment areas, into the lake. The crest level of such weirs was fixed with reference to the specified storage in the lake and the depth of flow over the weir was limited so as to increase the storage as much as possible.

Engineers are now making experiments by rounding off the top of the weir to increase the coefficient of discharge from 3·1 to 4 or 5 and thus to increase the discharge or reduce the depth of flow H and raise the crest level to increase the storage capacity of the lake. Attempts are being made now-a-days in some cases to dismantle the masonry at the top of the weir and round off the crest.

III.—*Barrages.*

Some barrages have been built across rivers in India and the one at Sukkur in Sindh across the river Indus, the largest and longest river in Hindustan, is the biggest in the world.

Sukkur barrage is built right across the river Indus. It is 4,725 feet long between the abutments and about 4,925 feet long from end to end of the over-bridge for working the gates. It consists of 66 spans each 60 feet, clear span. The piers are each 10 feet thick except the 7 abutment piers which are 25 feet thick. There are 5 scouring sluices on the right bank and 7 on the left bank. These are separated from the remaining 54 ordinary spans of the barrage by divided walls forming separate approach channels on each side of the river for supplying water to canals. These divide—walls parallel to the river banks run from Barrage to points about 500 feet upstream of the canal regulators on each bank.

There are 4 canals on the left bank and three on the right bank.

There are also guide banks on each side of the river both on the upstream and downstream of the barrage.

The Barrage gates have been supplied by Messrs. Ransomes and Rapiers and are each 60 feet clear span. The gates of the 54 ordinary spans of the barrage are $18\frac{1}{2}$ feet high, while those of the 12 scouring sluices are $22\frac{1}{2}$ ft. high. All gates are suspended and counterbalanced with weight, double their own, by means of ropes, thus reducing the height of the structure.

The gates are operated each, by separate gearings fixed on the high level bridge, spanning the piers. This gearing is operated by an electric motor, running on a special narrow gauge railway on the Bridge. Six separate Motors are provided, any one of which is able to operate any of the 66 gates.

The canals are provided with head regulators, the gates of which are also controlled by Motors.

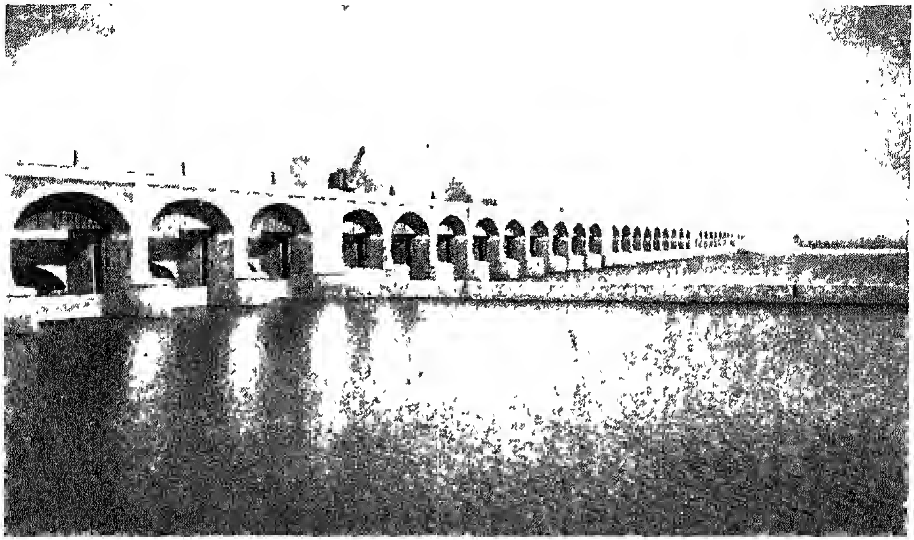
Two Photos (Fig. 100A and Fig. 100B) of the Sukkur barrage are given here.

For details of Kalabagh barrage, see P.E. Congress paper No. 251, volume 30 (1942).

For Emerson barrage at Trimmu, see paper No. 244 of P.E. Congress (1941), and also see publication No. 12 of Central Board of Irrigation, Government of India, by Khosla, Bose and Mckenzie-Taylor.

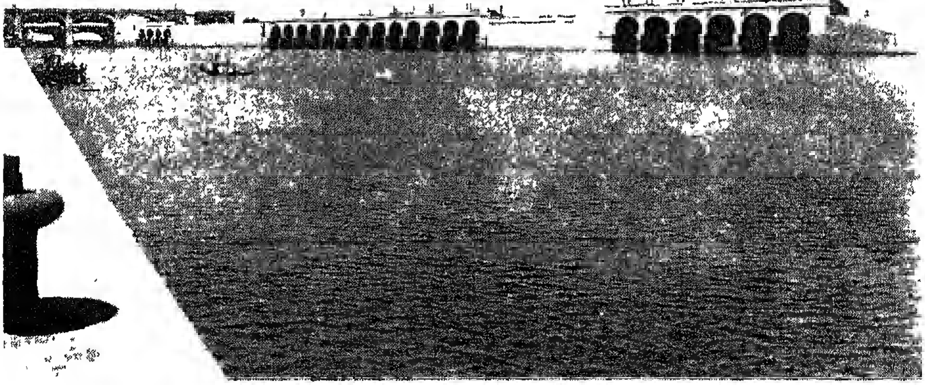
There are first class Irrigation research stations at Lahore, Amritsar, Malikpur, Poona, Karachi, Khadakwasla, Madras, etc. in India, worth visiting.

Fig. 100 A



Sukkur Barrage

Fig. 100 B



Head Regulators of Canals

Fig. 100 C

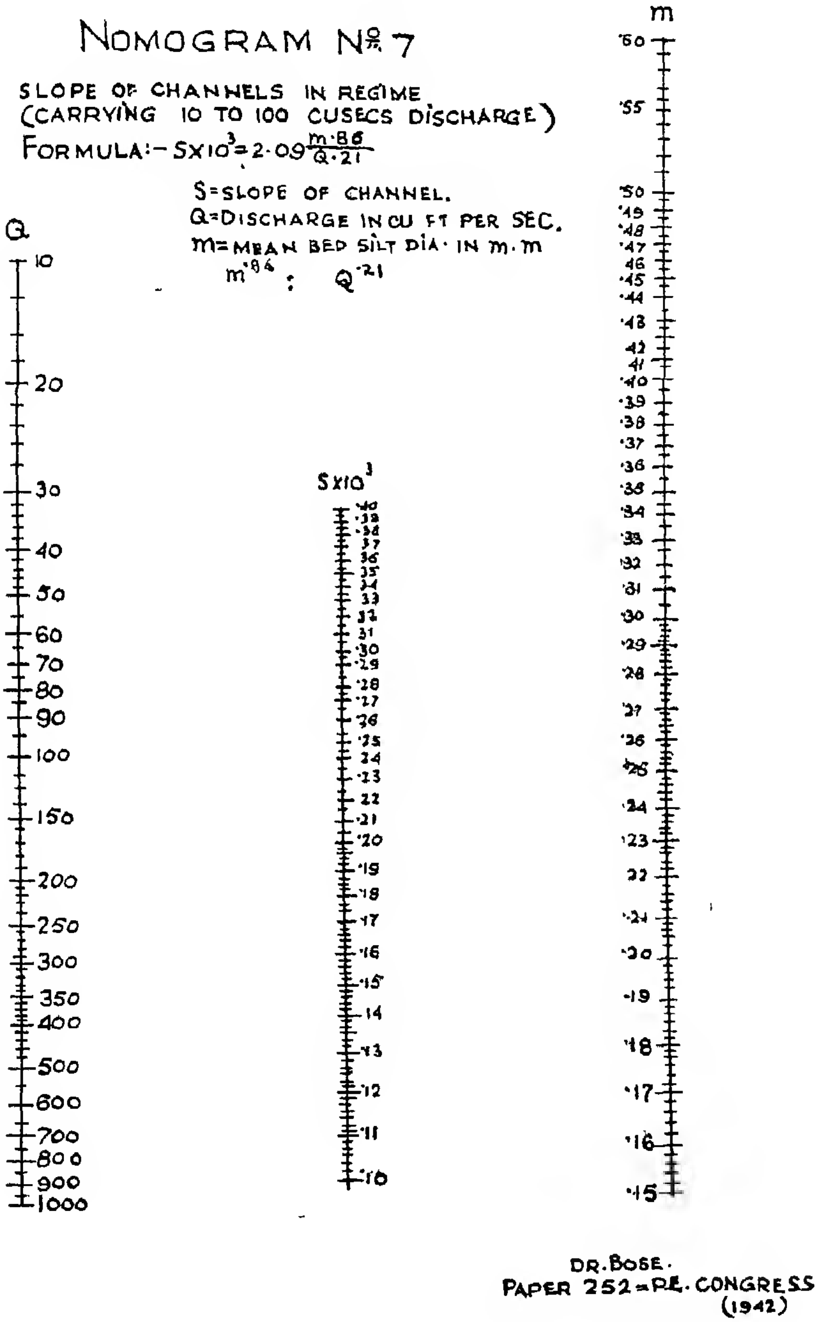

NOMOGRAM No. 7
SLOPE OF CHANNELS IN REGIME
(CARRYING 10 TO 100 CUSECS DISCHARGE)
FORMULA:- S×10³ = 2·09 m·86 / Q·21
S=SLOPE OF CHANNEL.
Q=DISCHARGE IN CU FT PER SEC.
m=MEAN BED SILT DIA· IN m·m
m·86 : Q·21
Q
S×10³
m
DR. BOSE.
PAPER 252 = P.E. CONGRESS
(1942)

Fig. 100 D

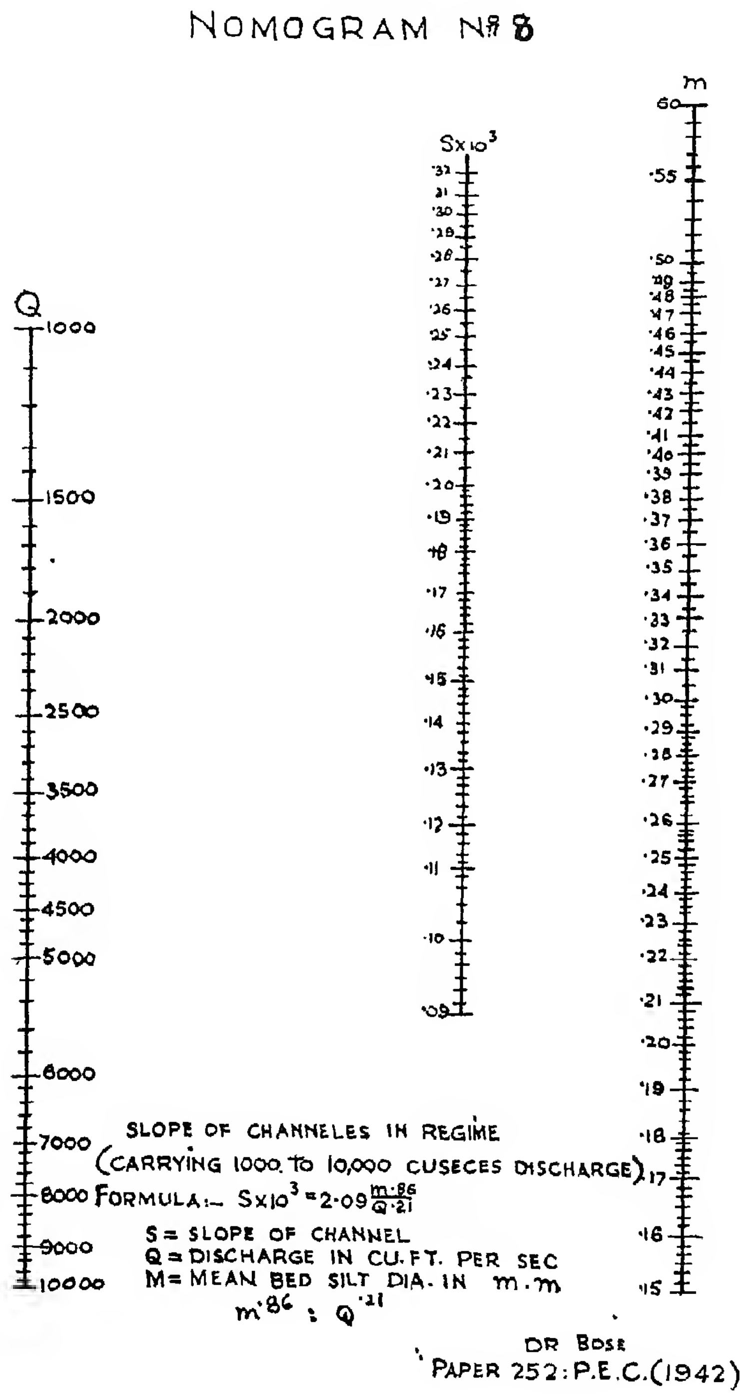
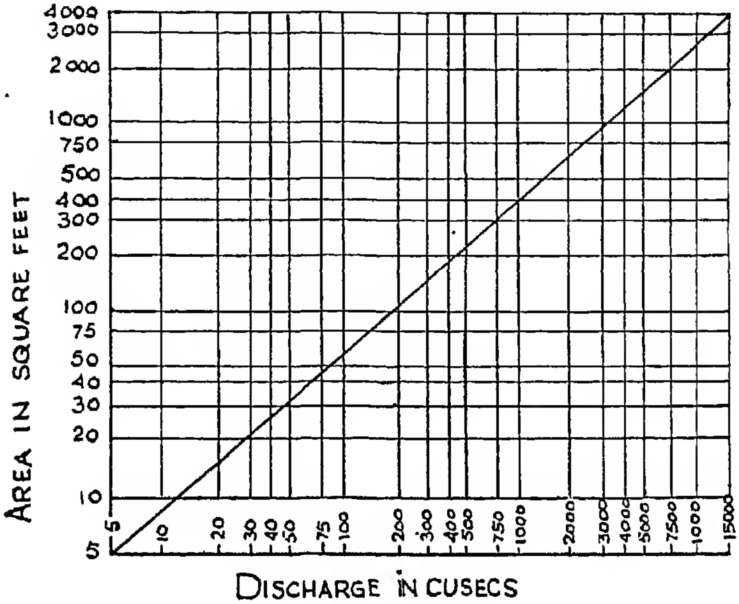
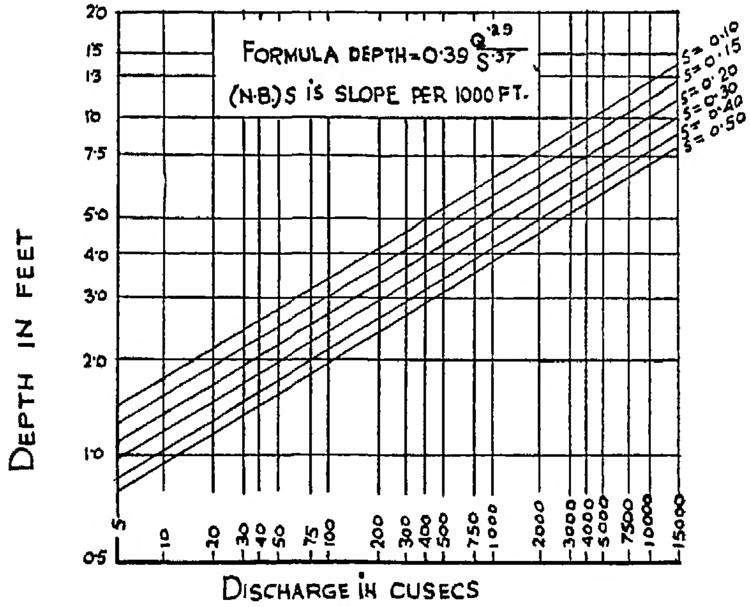

NOMOGRAM No 8
Q
1000
1500
2000
2500
3500
4000
4500
5000
6000
7000
8000
9000
10000
Sx10³
m
SLOPE OF CHANNELES IN REGIME
(CARRYING 1000 TO 10,000 CUSECES DISCHARGE)
FORMULA:— Sx10³ = 2·09 m·86 / Q·21
S = SLOPE OF CHANNEL
Q = DISCHARGE IN CU. FT. PER SEC
M = MEAN BED SILT DIA. IN m.m
m·86 : Q·21
DR BOSE
PAPER 252 : P.E.C. (1942)

Fig. 100 E

NOMOGRAM No. 9

FORMULA : AREA = $1.145\,Q^{.85}$





CHAPTER XV.

Drains and Sewers.

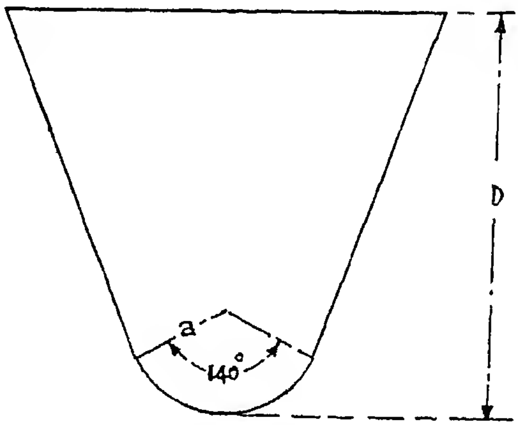
Wash water from bath-rooms, pantries and waste water from kitchens form what is called sullage. This is carried in towns by means of open drains to cultivate lands, or discharged in rivers.

When 'fæcal' matter and urine are added to sullage, then it is called sewage and is carried by sewers in towns to sewage disposal works.

112. *Open drains* :—These have been constructed so far of *pacca* bricks laid in lime mortar, cement pointed or cement plastered. The bricks generally absorb sullage water and stink badly, specially at night. Accordingly cement concrete blocks have been used lately for constructing open drains. These, if kept clean, are much better than brick drains whose use should be discouraged.

In the interests of public health, the dirty water should be removed by means of drains from the town as quickly as possible. This means that the velocity of flow in the drains should be very quick and no deposit should take place in the drain. The cross section of the drain and the slope should help in that way.

Fig. 100 F



A = Area

$$= 1{\cdot}585a^2 + 2{\cdot}128a(D-a) + {\cdot}364(D-a)^2$$

P = Wetted perimeter

$$= 3{\cdot}17a + 2{\cdot}128(D-a)$$

$$R = \text{Hydraulic Radius} = \frac{A}{P}$$

(*a*) *Types of open drains* :—Fig. 100 F shows the type of open drains suitable for the above purpose.

The cunette or the bottom of the drain is a segment of a circle, subtending an angle of 140° at the centre. The sides of the drain are tangents to the radii.

If d be the full depth of the drain and 'α' be the radius of the bottom part then A, the area of the drain

$= 1{\cdot}585\alpha^2 + 2{\cdot}128\alpha(d - \alpha) + {\cdot}364(d - \alpha)^2$ .... 99.

P, the wetted perimeter $= 3{\cdot}17\alpha + 2{\cdot}128(d - \alpha)$ .... 100.

$r = \text{Hydraulic mean radius} = \frac{A}{P}$

The bed slope of the drain should be such as to create self cleaning velocity not less than 2·5 feet per second. Nomogram No. 10, Fig. 103A gives particulars of grade, velocity, etc. Table No. XIV gives other particulars.

TABLE No. XIV.

Values in feet of R, hydraulic radius, and A, area, of Pegtop and semi-circular drains.

(M.E.S. HANDBOOK 1925, Vol. I, page 308).

Proportion of D, depth on invert to, α radius of base.		Radius, α, of segmental portion.								
		$1\frac{1}{2}''$	$2''$	$2\frac{1}{2}''$	$3''$	$3\frac{1}{2}''$	$4''$	$4\frac{1}{2}''$	$5''$	$6''$
$D = \alpha$	$R = \cdot 5\,\alpha =$	·0625	·0833	·104	·125	·146	·167	·188	·208	·25
	$A = 1\ 585\,\alpha^2 =$	0248	·0442	·069	·099	·135	·176	·223	·276	·297
$D = 2\alpha$	$R = \cdot 77\,\alpha =$	·0693	·129	·161	·193	·225	·257	·289	·322	·386
	$A = 4\cdot 077\alpha^2 =$	·0638	·1135	·177	·255	·347	·453	·572	·71	1·02
$D = 3\alpha$	$R = \cdot 982\alpha =$	·123	·163	·204	·246	·286	·327	·368	·408	·491
	$A = 7\cdot 297\,\alpha^2 =$	·114	·202	·316	455	·62	·81	1·03	1·37	1·825
$D = 4\alpha$	$R = 1\cdot 177\,\alpha =$	·147	·196	·245	·293	·342	·392	·44	·49	·588
	$A = 11\cdot 245\,\alpha^2 =$	·176	·313	·488	·702	·96	·135	1·58	1·95	2·82

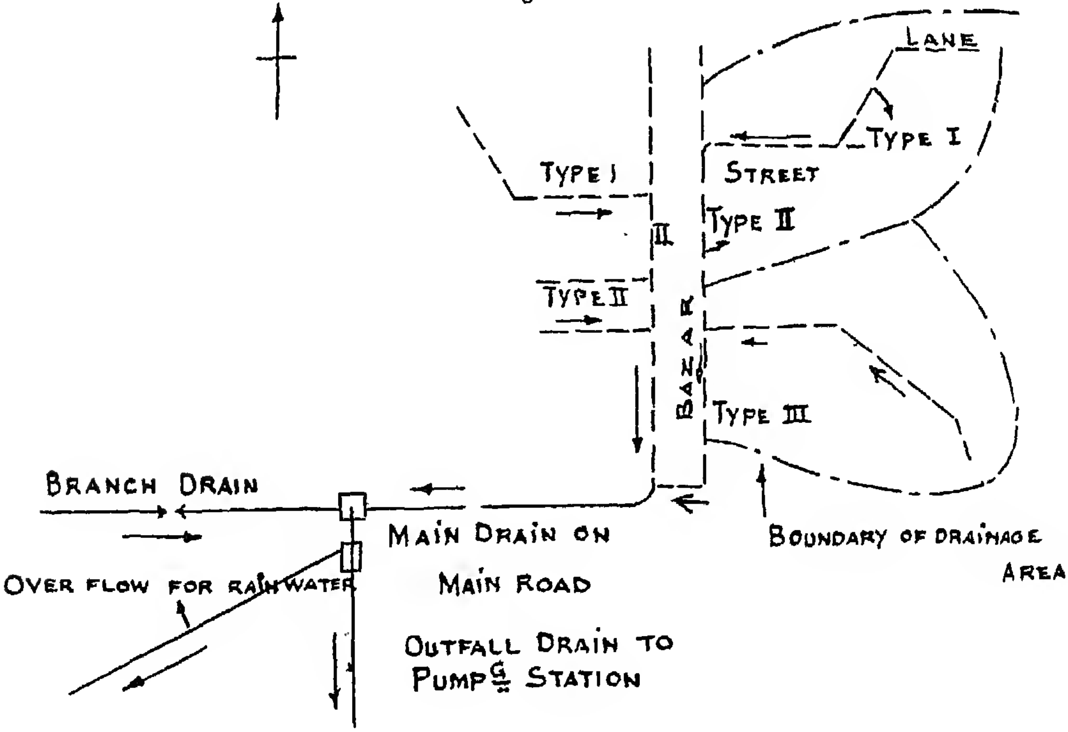
For semi-circular drains take as above with $D = \alpha$.

The public works department in different provinces have worked out type designs for open drains to carry different discharges.

Type No. I has a radius of 3 inches in the Cunette and depth of flow 3 to 4 inches. Type No. II has the same radius but the depth is increased by 3 inches and so on.

It is advisable to increase the radius of the Cunette from 3 to $4\frac{1}{2}$ inches (Type II revised) thence to 6 inches (Type III revised) as the discharges, the drains are required to carry, increase.

Fig. 101



b. Arrangement of open drains :—Fig. 101 shows an arrangement of open drains in Indian towns.

The sizes of drains depend upon the quantity of water supplied to the people by the municipal water-works or the quantity that they can draw from the wells, etc. Also the intensity of rainfall in the locality per hour is to be considered, as part of the rainfall enters these drains. The town area is to be divided into drainage blocks by means of spot levels in the streets, bazaars and roads as shown by the . – . – . lines in the figures. The area of each block is to be worked out in acres. A certain part of the rainfall per hour is to be adopted as the amount of expected flow in the drains. This depends upon the intensity of the rainfall in the locality. This expected flow includes sullage water and part of rain water. One inch rainfall per acre gives one cubic foot per second as discharge in the drain serving an area of one acre.

In hill stations, subject to heavy rainfalls, allowance of more than ¼ inch per hour is made, according to the amount of rainfall.

The smallest drain is type No. 1 with a radius of 3 inches in the Cunette. The depth is 3 inches to 4 inches but can be increased according to the difference of level between the invert of the drain and the street surface by brick-edging. This type of drain is laid along the middle line of narrow streets and types II and III are adopted, according to expected discharges and laid on the sides of main streets, to receive discharges from drains of type I.

Table No. XIV and nomogram No. 10 give particulars about open drains, their sizes, discharges, grades, and velocities. The available grades of streets, roads, and the country round about the town are to be utilized. If the available grades give velocity in the drains more than 2·5 feet per second, so much the better; otherwise suitable grades to produce self cleaning velocities are to be adopted. If the depth of excavation exceeds 4 feet on the sides of a road, the drain is to be covered; make it circular at least 24 inches in diameter or make it an ovoid sewer at least 20 inches × 30 inches.

If there be a piped water supply in the town flush tanks are to be installed at the heads of the drains to flush the same; otherwise a small portable centrifugal pump driven by a small petrol engine is to be moved from well to well to pump out water and flush the drain by means of a hose pipe. In no case flushing is to be neglected.

In very wide bazaars like those of Jaipur City, wide and shallow (saucer) drains are recommended.

TABLE No. XV.

Showing hydraulic mean depths, etc., of Punjab type drains of which profile measurements are given.

Type.	Section No.	Area.	Wetted perimeter	Hydraulic mean depth R	$\sqrt{R}$
I	(a)	·119	·87	·136	·369
I	(1) (1)	·093	·79	·118	·343
II	(2) (2)	·25	1·27	·197	·444
III	(3) (3)	·476	1·81	·263	·512
IV	(4) (4)	·75	2·35	·319	·565
V	(5) (5)	1·076	2·89	·37	·608
VI	(6) (6)	1·45	3·42	·42	·648

See Circular No. 1–721 dated 22nd October 1902 of Punjab Government, P.W.D.: B + Roads Branch.

Fig. 102

PROFILE-MEASUREMENTS OF DRAINS TO-DETERMINE AREAS

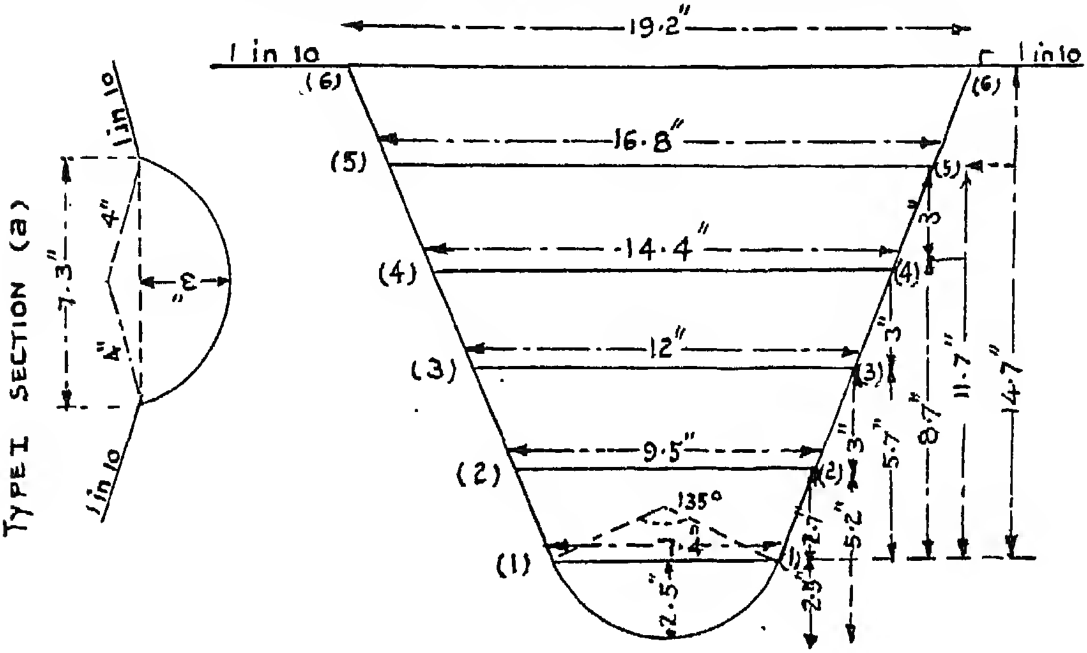


TABLE No. XVI.

Discharges (cusecs) and Velocities of Punjab Type Drains (1906).

V = Velocity ft./sec. D = Discharge.

Kutter's Coefficient N = ·015.

Similar to Fig. 102.

No. of Type drain.	SLOPE.									
	1 — 50		1 — 100		1 — 150		1 — 200		1 — 250	
	V	D	V	D	V	D	V	D	V	D
I	2·9	·27	2·0	·19	1·7	·16	1·4	·13	1·3	·12
I *a*	3·2	·38	2·3	·28	1·9	·23	1·6	·19	1·4	·17
II	4·3	1·17	3·1	·84	2·6	·71	2·2	·60	2·0	·54
III	5·4	2·70	3·9	1·95	3·1	1·55	2·7	1·35	2·4	1·20
IV	6·2	4·84	4·4	3·43	3·7	2·89	3·1	2·42	2·8	2·19
V	6·0	7·66	4·9	5·44	5·0	4·44	3·4	3·77	3·1	3·44
VI	7·5	10·88	5·4	7·83	4·4	6·38	3·8	5·51	3·5	5·07

TABLE No. XVI—*Contd.*

No. of Type drain.	SLOPE.											
	1 — 300		1 — 350		1 — 400		1 — 450		1 — 500		1 — 600	
	V	D	V	D	V	D	V	D	V	D	V	D
I ..	1·2	1·1	1·1	·10	1·0	·09	·9	·08	·8	·07	..	..
I *x* ..	1·3	·16	1 2	·14	1·1	·13	1·0	·12	·9	·11	..	..
II ..	1·8	·49	1·7	·46	1·6	·44	1·5	·41	1·4	·38	1·2	·33
III ..	2·2	1·10	2·1	1·05	1·9	·95	1·8	·90	1 7	·85	1·5	·75
IV ..	2·6	2·03	2·4	1·87	2·2	1·72	2·1	1·64	2·0	1·56	1·8	1·41
V ..	2·8	3·11	2·6	2·88	2·4	2·66	2·3	2·55	2·2	2·44	2·0	2·22
VI ..	3·1	4·50	2·9	4·20	2·7	3·91	2·5	3 62	2·4	3·48	2·2	3·19

(c) *Mr. Williams suggestions* :—Mr. George Bransby Williams, former Sanitary Engineer to the Government of Bengal, presented a paper to the Institution of Engineers (India), " Discharges of surface water drains and sewers in India " from which the following extracts are taken.

It has been generally accepted that the maximum run-off from any large town area will result from the maximum rate of rainfall that can continue for a period long enough for the flow from the most remote portion of the town area to reach its outlet. The time taken for this to occur is known as the " Period of concentration."

Experience shows that intensity of maximum possible rate of rainfall at any place varies inversely with the time for which it continues. It follows that, the larger the town area, the lower the maximum rate of run-off that need be allowed for. It is a bad practice to take a constant rate of run-off for any large town area. For a drainage scheme for a large town, the drains from small branches down to the main outfall drains, should be designed to take a progressively diminishing rate of run-off. In a system designed at a constant rate throughout, either the branch drains must be of inadequate capacity, or, else the main outfall drains must be unnecessarily large and expensive.

It may be stated as a general proposition that a surface drainage system should be capable of removing the discharge during rain storm of the maximum intensity that may be considered likely to occur during any period of a few consecutive years, sufficiently rapid to prevent its causing any serious damage or inconvenience.

$$I = \frac{3}{T + 0{\cdot}75} \qquad \ldots\ldots 101.$$

I = Intensity of rainfall in inches per hour.

T = Duration of the fall in hours.

Mr. Williams gives the above formula to work out the maximum intensity of rainfall in towns situated in Indian plains that may be reasonably expected to occur within a comparatively short period of years.

For period of concentration, *i.e.*, for the off-flow from the town area to reach the final outlet end of the town, Mr. Williams gives the following formula for a municipal drainage area situated on an Indian plain in which the ground is more or less flat and the period of concentration depends upon the artificial drainage channels.

$$t = 5 \times \sqrt[5]{A^2} \qquad \ldots\ldots 102,$$

t = period of concentration in minutes.

A = the drainage area in acres.

Regarding discharging capacity of drains Mr. Williams advises as under.

Not all the rain that falls on the town area, will reach the outfall drain; some will be absorbed by the ground, some will fall into the tank or be evaporated by the surface of the ground. The percentage of the total fall that will eventually flow off will obviously depend upon local conditions. In an urban area on the Indian plains, it will probably be about 85 per cent. In a suburban or semirural area, it is likely to be about 65 per cent while in a rural area it will be less.

Drainage schemes for Urban areas in Indian plains should be designed to carry an off-flow at the rate of one quarter of the maximum probable rate of rainfall per hour during the period of concentration on the area served.

TABLE No. XVII.

Table showing Maximum Probable Intensities of Rainfall and Rates of run-off to be provided for in Urban and Suburban Areas.

Area.	Period of concentration.	Probable maximum intensity of rainfall.	Rate of Run-off to be provided for.	
			Urban Area.	Suburban Area.
Acres.	Minutes.	Inches per hour.	Inches per hour.	Inches per hour.
5	10	3·35	·84	·67
10	13	3·11	·78	·62
25	28	2·87	·72	·57
50	24	2·62	·65	·52
75	28	2·47	·62	·49
100	32	2·33	·58	·47
200	42	2·07	·51	·41
500	60	1·72	·43	·34
1000	79	1·46	·36	·29
1500	93	1·32	·32	·26
2000	104	1·20	·29	·23
2500	114	1·12	·27	·21
3000	123	1·07	·25	·20
4000	138	·98	·23	·19
5000	151	·92	·21	·17

(*d*) *Suggestions from Bombay P.W.D. Hand-book* :—P. W. D. Hand-book, Bombay, Vol. II, Eighth edition (1931) gives following suggestions on sullage and storm-water surface drains.

Surface drains are often constructed with square bottoms and of uniform section; the result being that at the time of the minimum flow the velocity is insufficient and silt deposit takes place, and if it is not properly attended to, the drains get choked up. Therefore, in designing the section of an open drain, the minimum as well as the maximum discharge must be considered, and a self-cleansing velocity for the minimum discharge must be secured.

Preliminary calculation :—Every surface drain will be expected to carry all the sullage water and a large portion of the storm-water. Therefore the quantity of sullage water (which will be equal to the water-supply per head per day) from a particular section should be first considered, and the section of the invert portion should be so designed as to carry away in 6 hours 50 per cent of the average daily supply, without leaving any deposit as far as possible. Next, the area which will be drained by the particular section of the drain should be found out, and a suitable run-off depending on the intensity of the rainfall at the place, and also the nature of the premises, should be assumed and the quantity of the storm-water to be passed down should be calculated and added to the quantity of sullage previously found out ; and a complete section with the invert as already fixed should be designed.

N.B.—A run-off from ½″ to 1″ per hour may be taken according to the rainfall at the place.

In determining the size of a drain attention should be paid to securing a self-cleansing velocity, *i.e.*, the minimum velocity obtained for any section should not be less than 3 feet per second. Though strictly speaking surface drains should not be covered over ; in most towns it is absolutely impossible to avoid covered drains. The aim should, however, be to reduce the length of covered drains, as far as possible because the covered portions generally harbour rats and they are often neglected.

The following rule regarding the length of covered drains will be useful :—

When the gutter covered is level with the road surface at the edge of the gutter, no covered length should exceed 3 feet. No covering to a gutter should be permitted within 4 feet of the next covering. The minimum clearance between the gutter covering and the invert level of the gutter should be one foot.

The inside of the gutter is generally rendered with cement on the invert ; but this rendering is damaged by the rough scraper, generally used by the sweepers while cleaning. The repairs in such places are often neglected.

It will, therefore, be always advisable to have the invert of glazed stoneware pipes. The type sections recommended for surface drains are of the rounded V type (*see* Fig. 100F). This shape is suitable because the area of the section decreases in proportion to the decrease of the discharge, and a uniform velocity is generally maintained.

For further details see " Surface Drainage " by H. A. Gubbay, A.M.I.C.E.

113. *Sewers.*—Underground pipes, draining more than one property and carrying 'fæcal' matter, urine, and waste water are termed sewers.

a. Size of sewers :—In New Delhi 30 gallons per head per day of 24 hours was allowed as inflow in the sewers. Thus the average dry weather flow $= \dfrac{30}{24 \times 60}$ lons per minute per head of population. The maximum flow was taken as twice the average, the stoneware pipe sewers were designed to carry 6 times the average dry weather flow when running full and the brick sewers to carry this flow when running three-quarters full.

The water-works were designed to supply 30 gallons per head per day of 24 hours, to each individual, the supply being open for 24 hours. This is the standard for most towns in India where sewerage system is provided or contemplated for the town.

The greater part of the Calcutta sewerage system was designed to carry a run-off of ¼ inch per hour during a rainfall. This was a mistake as the sewers are too big and expensive.

In New Delhi, separate surface drains are provided alongside of the roads for rainwater. Where these drains become too wide and too deep, they become covered underground drains, circular in section.

Every attempt is made to exclude rainwater from the sewers.

It must be borne in mind that when the sewers carry the maximum flow (= twice the average dry weather flow), the depth of flow in the sewer should be such as to produce a self cleansing velocity of not less than 2.5 feet per second, otherwise silt deposit will be formed in the bed of the sewer. Preferably the depth of flow in that case should be at least one-quarter of the full depth.

Table Nos. XVIII and XIX give diameters of sewers, grades, and velocities.

In New Delhi, the pipe sewers and brick sewers were provided with automatic flushing tanks.

The writer was employed on the design and construction of sewerage scheme for New Delhi for 14 years (1913-1927).

b. Velocity and gradients :—The following table gives the minimum gradients for sewers and limiting permissible velocities. The minimum gradients are selected to give a self cleansing velocity of $2\frac{1}{2}$ feet per second or thereabout, in a sewer flowing half full without recourse to flushing; with flushing lower gradients are now accepted as permissible.

Every attempt should be made to so design as to attain a velocity of 3 to $3\frac{1}{2}$ feet per second, without flushing:—

TABLE No. XVIII.

Sewers—Grades—Limiting Velocities.

Diameter of sewer.	Minimum gradients.		Limiting velocities.
	Without flushing.	With flushing.	
4″ ..	1 in 40	....	No limit.
6″ ..	1 in 100	1 in 175	10 ft. per second.
7″ ..	1 in 125	1 in 200	9 ,, ,,
9″ ..	1 in 185	1 in 275	9 ,, ,,
12″ ..	1 in 270	1 in 385	8 ,, ,,
15″ ..	1 in 410	1 in 500	6 ,, ,,
18″ ..	1 in 600	1 in 600	6 ,, ,,
24″ ..	1 in 960	....	6 ,, ,,
30″ ..	1 in 1300	....	6 ,, ,,

NOTE:—For 4 inches house connections no limiting velocity need be specified. For circular pipe sewers 19 to 45 inches in diameter minimum gradient should not be less than 1 in 600, where possible.

Inclination of pipes for special velocities.

The following table gives the falls required to produce certain velocities in pipes of different sizes when running full or half full, where the coefficient of roughness, N, is equal to ·013 :—

TABLE No. XIX.

Diameter of pipe.	Velocity in feet per second.						
	2	2½	2¾	3	4	5	6
Inches.	Sine of inclination (1 over) :—						
3	50	32	26	22	12	8	5
4	82	52	43	36	20	13	9
6	155	100	84	70	39	25	17
9	295	190	155	130	75	48	33
12	460	295	245	205	115	75	52
15	640	415	345	290	160	105	73
18	840	540	450	375	210	135	95
24	1250	820	680	570	325	205	145

Type, velocity, grade and discharge.—In New Delhi salt glazed stoneware pipes were used upto 18 inches diameter. No reinforced cement concrete pipes were used. In Karachi, also, no cement concrete pipes are used as sewers. Mr. D. A. Howell, the Chief Engineer for Public Health Engineering Department, Punjab (1945) used cement concrete pipes on some roads in Lahore for the sewerage scheme.

In New Delhi, the sewers above 18 inches in diameter have been built of bricks laid in lime mortar and cement plastered on inside, right up to 84 inches diameter. Ovoid sewers were built in one or two cases only.

In Madras city circular sewers have been built lately. I have seen boys crawling through 24 inches sewers and young men crawling through 36 inches circular sewers for cleaning purposes. It is more comfortable to walk through 4 feet × 6 feet ovoid sewer for cleaning purposes than through 4 feet diameter circular sewer.

Egg-shaped or oval sewers were freely built in Europe in 19th century. The depth of flow in a sewer fluctuates in every 24 hours, therefore in order that the stream may be of the best form or nearly so, when the sewage level is low, and deposits may not occur, an oval form was adopted.

The self cleansing velocity is not appreciably reduced with the fall in the depth of flow. The velocity with the sewer one-third full is three-fourths of the velocity when it is two-thirds full.

TABLE No. XX.

Self cleansing velocity for circular pipes and sewers.

Diameter of sewer in inches.	Safe self cleansing velocity : feet per second.	
	When full	When two-thirds full
Up to 9	3·4	3
9 to 24	2 8	2·5
over 24	2·2	2

House drains should not have a steeper slope than 1/10, or iron pipes must be used, or the drains stepped with man-holes. If sufficient fall to give the minimum self cleansing velocity is not available, flushing arrangements are necessary.

On the other hand, the maximum safe velocity for stoneware, brick or concrete sewers is 4·5 feet per second.

At present the tendency seems to be to build circular sewers only and as far as the writer knows, no extensive system of egg-shaped sewers has been constructed for several years.

Owing to its peculiar shape, an egg-shaped sewer gives higher velocities than a circular sewer at low discharges. It is sufficient to provide a velocity of 2·5 feet per second when one-third full and in the case of large sewers over 4 feet in transverse diameter 2 5 feet per second when flowing two-thirds full.

TABLE No. XXI.

Showing slopes required for circular sewers running full to generate minimum self cleansing and maximum permissible velocities (n = ·013).

Diameter of circular sewer in inches.	Minimum.		Maximum.	
	Self cleansing velocity.	Slope in parts per thousand.	Permissible safe velocity.	Slope in parts per thousand.
House drains ..	3·4	35 to 10	5	100 to 20
9	3·4	9·6	4·5	17
12	3·2	5·5	4·5	11
18	3·0	2·6	4·5	5·8
24	2·8	1·5	4·5	3·9
30	2·6	·97	4·5	2·8
36	2·4	·65	4·5	2·2
48	2·2	·37	4·5	1·5
60	2·2	·28	4·5	1·1
72	2·2	·22	4·5	·87
84	2·2	·18	4·5	·71

TABLE No. XXII.

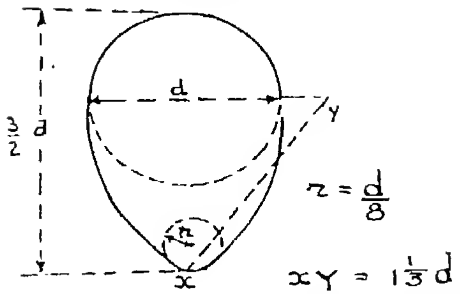
For good Pattern Egg-shaped Sewers. (*Fig.* 103). (Garrett 1909).

Radius of Top.	Dimensions of sewer in inches.	Area of sewer in square feet.	Brick work in cubic yards per yard run.		
			4½ inch work.	9 inch work.	13½ inch work.
Inches					
6	12 × 18	1·115	·2124	·5231	....
7	14 × 21	1·517	·2396	·5775	....
8	16 × 24	1·982	·2669	·6320	....
9	18 × 27	2 508	·2941	·6864	....
10	20 × 30	3·097	·3213	·7409	....
11	22 × 33	3·747	·3486	·7953	....
12	24 × 36	4·460	·3758	·8498	....
13	26 × 39	5·234	·4030	·9042	....
14	28 × 42	6·070	·4302	·9587	....
15	30 × 45	6·968	·4574	1·0131	....
16	32 × 48	7·929	·4847	1·0685	....
17	34 × 51	8·951	·5119	1·1220	....
18	36 × 54	10·035	·5392	1·1764	....
19	38 × 57	11·181	·5664	1·2309	....
20	40 × 60	12·387	·5936	1·2853	2·0753
21	42 × 63	13·659	....	1·3398	2·1569
22	44 × 66	14·991	....	1·3942	2·2386
23	46 × 69	16·384	....	1·4487	2·3203
24	48 × 72	17·840	....	1·5032	2·4020
25	50 × 75	19·358	....	1·5576	2·4837
26	52 × 78	20·938	....	1·6120	2·5654
27	54 × 81	22·579	....	1·6665	2·6470
28	56 × 84	24·283	....	1·7209	2·7287
29	58 × 87	26·048	....	1·7754	2·8104
30	60 × 90	27·875	....	1·8299	2·8920
31	62 × 93	29·765	....	1·8843	2·9737
32	64 × 96	31·716	....	1·9387	3·0554
33	66 × 99	33·730	....	1·9932	3·1371
34	68 × 102	35·805	....	2·0477	3·2188
35	70 × 105	37·942	....	2·1021	3·3001
36	72 × 108	40·141	....	2·1566	3·3821

TABLE No. XXIII.

Areas, etc. of Egg-shaped Sewers. (Garrett 1909).
Egg-shape—good form.

Fig. 103



		Full	$\frac{2}{3}$ full	$\frac{1}{3}$ full
B	Area of section	$1 \cdot 1150\ d^2$	$\cdot 7223\ d^2$	$\cdot 2543\ d^2$
	Perimeter wetted	3·9205 d	2·3497 d	1·3246 d
	Hydraulic mean depth	·2844 d	·3074 d	·1920 d

The above form Fig. 103 is said to be the best due to the maintenance of a high comparative velocity at low discharges ; and is also strong and resists the pressure of earth well.

Owing to its peculiar shape, an egg-shaped sewer gives higher velocities than a circular sewer at low discharges.

TABLE No. XXIV.

The following table gives the grades adopted while building sewers in different cities, to produce self-cleaning velocity 2·5 feet per second.

Grades : 1 in.

Diameter.	New Delhi.		Lahore.		Karachi.	
	Max.	Mini.	—	Mini.	—	Mini.
4″	20	40	..	..	..	..
6″	40	135	..	135	..	100
9″	60	230	..	230	..	185
12″	90	340	..	340	..	270
15″	130	460	..	460	..	410
18″	160	580	..	..	..	600
21″	200	720	..	..	..	800
24″	230	860	..	..	..	960
30″	..	..	..	..	..	1300

For discharges through circular sewers and oval sewers M/S Santo Crimp and Ernest Bruges have prepared very good tables based on their formula.

$V = 124 \sqrt[3]{r^2} \sqrt{s}.$ V = Velocity in feet per sec.
r = Hydraulic mean depth in feet.
s = Ratio of fall to length.

Table No. XXV.

Proportional Velocities and Discharges, in circular sewers and pipes running partly full (Calculated from the value of $\sqrt[3]{r^2}$) Santo Crimp's tables (1897).

1	2	3	4	5	6
Prop. Depth.	Prop. Area.	Prop. H. M. D.	Prop. Velocity.	Prop. Discharge.	Prop Depth.
·01	·0017	·0265	·0890	·00015	·01
·02	·0048	·0528	·1408	·0007	·02
·03	·0087	·0789	·1839	·0016	·03
·04	·0134	·1047	·2221	·0030	·04
·05	·0187	·1302	·2569	·0048	·05
·06	·0245	1555	·2892	·0071	·06
·07	·0308	·1805	·3194	·0098	07
·08	·0375	·2053	·3481	·0130	08
·09	·0446	·2298	·3752	·0167	09
·10	·0520	·2541	·4012	·0209	·10
·11	·0598	·2781	4260	·0255	11
·12	·0680	·3018	·4500	·0306	·12
·13	·0764	·3253	·4730	·0361	·13
·14	·0851	·3485	·4953	·0421	·14
·15	·0941	·3715	·5168	·0486	·15
·16	·1033	·3942	·5376	·0555	·16
·17	·1127	·4167	·5578	·0629	·17
·18	·1224	·4388	·5775	·0707	·18
·19	·1323	·4607	·6965	·0789	·19
·20	·1424	·4824	·6151	·0876	·20
·21	·1527	·5037	·6331	·0966	·21
·22	·1631	·5248	·6507	·1062	·22
·23	·1737	·5457	·6678	·1160	·23
·24	·1845	·5662	·6844	·1263	·24
·25	·1955	·5865	·7007	·1370	·25
·26	·2066	·6065	·7165	·1480	·26
·27	·2178	·6262	·7320	·1594	·27
·28	·2292	·6457	·7470	·1712	·28
·29	·2407	·6649	·7618	·1834	·29
·30	·2523	·6838	·7761	·1958	·30
·31	·2640	·7024	·7901	·2086	·31
·32	·2759	·7207	·8038	·2217	·32
·33	·2878	·7387	·8172	·2352	·33
·34	·2998	·7565	·8302	·2489	·34
·35	·3119	·7740	·8430	·2629	·35
·36	·3241	·7911	·8554	·2772	·36
·37	·3364	·8080	·8675	·2918	·37
·38	·3487	·8246	·8794	·3066	·38
·39	·3611	·8409	·8909	·3217	·39
·40	·3735	·8569	·9022	·3370	·40
·41	·3860	·8726	·9132	·3525	·41
·42	·3986	·8881	·9239	·3682	·42
·43	·4112	·9031	·9343	·3841	·43
·44	·4238	·9179	·9445	·4003	·44
·45	·4364	·9323	·9544	·4165	·45
·46	·4491	·9465	·9640	·4330	·46
·47	·4618	·9604	·9734	·4495	·47
·48	·4745	·9739	·9825	·4662	·48
·49	·4873	·9871	·9914	·4831	·49
·50	·5000	1·0000	1·0000	·5000	·50

TABLE No. XXV—*Contd.*

1	2	3	4	5	6
Prop. Depth.	Prop. Area.	Prop. H. M. D.	Prop. Velocity.	Prop. Discharge	Prop. Depth.
·51	·5127	1 0126	1·0084	·5170	·51
·52	·5255	1·0249	1·0165	·5341	·52
·53	·5382	1·0367	1·0243	·5513	·53
·54	·5509	1·0483	1 0319	·5685	·54
·55	·5636	1 0595	1·0393	·5857	·55
·56	·5762	1·0704	1·0464	·6030	·56
·57	·5888	1·0810	1·0533	·6202	·57
·58	·6014	1·0911	1·0599	·6374	·58
·59	·6140	1·1010	1·0663	·6546	·59
·60	·6265	1·1106	1·0724	·6718	·60
·61	·6389	1·1197	1·0783	·6889	·61
·62	·6513	1·1285	1·0839	·7060	·62
·63	·6636	1·1368	1·0893	·7229	·63
·64	·6759	1·1449	1·0944	·7397	·64
·65	·6881	1·1526	1·0993	·7564	·65
·66	·7002	1·1599	1·1039	·7730	·66
·67	·7122	1·1667	1·1083	·7893	·67
·68	·7241	1·1732	1·1124	·8055	·68
·69	·7360	1·1793	1·1162	·8215	·69
·70	·7477	1·1849	1·1198	·8372	·70
·71	·7593	1·1902	1·1231	·8527	·71
·72	·7708	1·1950	1·1261	·8680	·72
·73	·7822	1·1994	1·1288	·8829	·73
·74	·7934	1·2033	1·1313	·8976	·74
·75	·8045	1·2068	1·1335	·9119	·75
·76	·8155	1·2097	1·1353	·9258	·76
·77	·8263	1·2123	1·1369	·9394	·77
·78	·8309	1·2143	1·1382	·9524	·78
·79	·8473	1·2158	1·1391	·9652	·79
·80	·8576	1·2168	1·1397	·9775	·80
·81	·8677	1 2172	1 1400	·9892	·81
·82	·8776	1 2170	1·1399	1·0004	·82
·83	·8873	1·2164	1·1395	1·0110	·83
·84	·8967	1·2150	1·1387	1·0211	·84
·85	·9059	1·2131	1·1374	1·0304	·85
·86	·9149	1·2104	1 1358	1·0391	·86
·87	·9236	1·2070	1·1337	1·0471	·87
·88	·9320	1·2029	1·1311	1·0542	·88
·89	·9402	1·1980	1·1280	1·0605	·89
·90	·9480	1·1921	1·1243	1·0658	·90
·91	·9554	1·1853	1·1200	1·0701	·91
·92	·9625	1·1774	1·1150	1·0732	·92
·93	·9692	1·1684	1·1093	1·0752	·93
·94	·9755	1·1579	1·1027	1·0757	·94
·95	·9813	1·1458	1·0950	1·0745	·95
·96	·9866	1·1316	1·0859	1·0714	·96
·97	·9913	1·1148	1·0751	1·0657	·97
·98	·9952	1·0941	1·0618	1·0567	·98
·99	·9983	1·0663	1·0437	1·0419	·99
1·00	1 0000	1·0000	1·0000	1·0000	1·00

TABLE No. XXVI.

The Areas of egg-shaped sewers in square feet and the value of R and $\sqrt[3]{R^2}$
(R = Hydraulic mean depth in feet).
(Tables and Diagrams by Santo Crimp, C.E., Page 99 (1897).

Size.	Full.		$\sqrt[3]{R^2}$
	Area in square feet.	R, in feet.	
Ft. in. Ft. in.			
1–8 × 2–6	3·1903	·4828	·6154
1–10 × 2–9	3·8602	·5311	·6558
2–0 × 3–0	4·5940	·5794	·6950
2–6 × 3–9	7·1781	·7242	·8064
3–0 × 4–6	10·337	·8691	·9107
3–4 × 5–0	12·761	·9657	·9770
3–6 × 5–3	14·069	1·0139	1·0092
3–8 × 5–6	15·441	1·0622	1·0410
4–0 × 6–0	18·376	1·1588	1·1032
4–2 × 6–3	19·939	1·2071	1·1337
5–0 × 7–6	28·712	1·4485	1·2802
5–4 × 8–0	32·668	1·5451	1·3365
5–10 × 8–9	39·081	1·6899	1·4188
6–0 × 9–0	41·346	1·7382	1·4457

Size.	Two-thirds Full		$\sqrt[3]{R^2}$
	Area in square feet.	R, in feet.	
Ft. in. Ft. in.			
1–8 × 2–6	2·0994	·5262	·6518
1–10 × 2–9	2·5402	·5788	·6945
2–0 × 3–0	3·0232	·6314	·7360
2–6 × 3–9	4·7237	·7892	·8540
3–0 × 4–6	6·8022	·9471	·9644
3–4 × 5–0	8·3978	1·0523	1·0346
3–6 × 5–3	9·2585	1·1049	1·0688
3–8 × 5–6	10·161	1·1576	1·1025
4–0 × 6–0	12·093	1·2628	1·1683
4–2 × 6–3	13·121	1·3154	1·2005
5–0 × 7–6	18·895	1·5784	1·3557
5–4 × 8–0	21·498	1·6836	1·4152
5–10 × 8–9	25·718	1·8416	1·5024
6–0 × 9–0	27·209	1·8942	1·5309

TABLE No. XXVI—*Contd.*

Size.	One-third Full		$\sqrt[3]{R^2}$
	Area in square feet.	R, in feet.	
Ft. in. Ft. in.			
1–8 × 2–6	·7889	·3443	·4912
1–10 × 2–9	·9545	·3787	·5233
2–0 × 3–0	1·1360	·4132	·5548
2–6 × 3–9	1·7750	·5165	·6437
3–0 × 4–6	2·5560	·6198	·7269
3–4 × 5–0	3·1556	·6887	·7799
3–6 × 5–3	3·4790	·7231	·8056
3–8 × 5–6	3·8182	·7575	·8310
4–0 × 6–0	4·5440	·8264	·8806
4–2 × 6–3	4·9306	·8608	·9049
5–0 × 7–6	7·1000	1·0330	1·0219
5–4 × 8–0	8·0782	1·1019	1·0668
5–10 × 8–9	9·6639	1·2052	1·1325
6–0 × 9–0	10·224	1·2396	1·1540

For proportional areas and hydraulic mean depth when flowing at other proportional depths see table 27.

TABLE No. XXVII.

Proportional values for velocities and discharges in egg-shaped sewers running partly full.

(Calculated from the value of $3\sqrt{r^2}$) (Santo Crimp's Tables 1897).

Prop. Depth.	Prop. Area.	Prop. H. M. D.	Prop. Velocity.	Prop. Discharge.
·01	·0015	·0340	·1050	·0002
·02	·0042	·0671	·1651	·0007
·03	·0076	·0092	·2143	·0016
·04	·0116	·1302	·2569	·0030
·05	·0161	·1603	·2951	·0048
·06	·0209	·1894	·3298	·0070
·07	·0260	·2171	·3612	·0094
·08	·0315	·2435	·3899	·0123
·09	·0372	·2679	·4158	·0154
·10	·0432	·2917	·4399	·0190

TABLE No. XXVII—*Contd.*

Prop. Depth.	Prop. Area.	Prop. H M D.	Prop. Velocity.	Prop. Discharge
·11	·0495	·3149	·4628	·0229
·12	·0561	·3375	·4847	·0272
·13	·0629	·3591	·5051	·0318
·14	·0700	·3802	·5249	·0368
·15	·0773	·4009	·5437	·0421
·16	·0849	·4208	·5615	·0477
·17	·0927	·4403	·5787	·0536
·18	·1007	·4594	·5954	·0599
·19	·1089	·4781	·6115	·0666
·20	·1173	·4962	·6268	·0735
·21	·1259	·5139	·6416	·0808
·22	·1347	·5314	·6561	·0884
·23	·1437	·5488	·6703	·0963
·24	·1530	·5660	·6842	·1046
·25	·1625	·5829	·6976	·1133
·26	·1721	·5995	·7108	·1223
·27	·1818	·6158	·7238	·1316
·28	·1917	·6319	·7364	·1412
·29	·2017	·6478	·7485	·1510
·30	·2119	·6633	·7605	·1610
·31	·2224	·6787	·7723	·1714
·32	2331	·6938	·7837	·1822
·33	·2438	·7086	·7948	·1934
·34	·2546	·7233	·8056	·2049
·35	·2655	·7378	·8162	2166
·36	·2766	·7519	·8267	·2286
·37	·2879	·7658	·8370	·2410
·38	·2993	·7795	·8470	·2535
·39	·3108	·7931	·8569	·2663
·40	·3223	·8066	·8666	·2793
·41	·3339	·8199	·8760	·2925
·42	·3457	·8328	·8852	·3061
·43	·3577	·8456	·8942	·3199
·44	·3697	·8582	·9031	·3339
·45	·3818	·8705	·9117	·3481
·46	·3940	·8827	·9202	·3625
·47	·4062	·8947	·9285	·3772
·48	·4186	·9064	·9366	·3920
·49	·4310	·9180	·9446	·4071
·50	·4434	·9294	·9523	·4223
·51	·4560	·9405	·9599	·4377
·52	·4686	·9515	·9674	·4533
·53	·4813	·9623	·9747	·4691
·54	·4940	·9729	·9819	·4850
·55	·5068	·9834	·9885	·5011
·56	·5196	·9936	·9956	·5173
·57	·5324	1·0035	1·0022	·5336
·58	·5453	1·0132	1·0087	·5501
·59	·5583	1·0228	1·0151	·5667
·60	·5712	1·0323	1·0214	·5835
·61	·5842	1·0415	1·0275	·6004
·62	·5972	1·0504	1·0334	·6174
·63	·6102	1·0593	1·0391	·6346

TABLE No. XXVII—*Contd.*

Prop Depth	Prop. Area.	Prop H M. D.	Prop. Velocity.	Prop. Discharge.
·64	·6233	1 0680	1·0447	·6513
·65	·6363	1·0763	1·0502	·6683
·66	·6494	1·0845	1·0556	·6855
·67	·6624	1·0925	1·0608	·7027
·68	·6755	1·1003	1·0658	·7199
·69	·6885	1·1079	1·0707	·7372
70	·7015	1·1152	1·0754	·7544
·71	·7145	1·1222	1·0799	·7716
·72	·7274	1·1289	1·0842	·7887
·73	·7403	1·1354	1·0883	·8057
·74	·7531	1·1415	1·0922	·8225
·75	·7658	1·1471	1·0952	·8392
·76	·7784	1·1525	1·0992	·8556
·77	·7909	1·1574	1·1023	·8718
·78	·8032	1·1618	1·1051	·8878
·79	·8153	1·1661	1·1077	·9036
·80	·8273	1·1696	1·1100	·9191
·81	·8392	1·1724	1·1119	·9332
·82	·8510	1·1748	1·1134	·9475
·83	·8625	1·1767	1·1146	·9614
·84	·8738	1·1780	1·1155	·9747
·85	·8848	1·1787	1·1159	·9873
·86	·8955	1·1783	1·1156	·9993
·87	·9060	1·1778	1·1153	1·0106
·88	·9163	1·1763	1·1144	1·0210
·89	·9262	1·1739	1·1128	1·0306
·90	·9357	1·1704	1·1106	1·0392
·91	·9448	1·1661	1·1078	1·0467
·92	·9535	1·1606	1·1043	1·0531
·93	·9618	1·1538	1·1000	1·0580
·94	·9695	1·1456	1·0948	1·0615
·95	·9767	1·1356	1·0885	1·0631
·96	·9832	1·1235	1·0807	1·0626
·97	·9891	1·1086	1·0710	1·0594
·98	·9940	1·0897	1·0590	1·0526
·99	·9979	1·0638	1·0421	1·0399
1 00	1·0000	1·0000	1·0000	1·0000

113 (C) :—*Sullage and sewage Pumping Stations* :—

Ancients were wise to build an intercepting open drain round the walled city of Lahore, with a cunette in the bottom to carry sullage and the upper part to carry storm water, right into the river Ravi by Gravity. The part of the out-fall through the old bed of the river was covered and provided with Manholes here and there from which the Sullage was lifted by Persian Wheels, driven by bullocks and then mixed with fresh water from shallow wells and used for sullage Farms. This arrangement brought lot of income to the Lahore Municipality without the use of any Pumps.

As the city of Lahore has now extended far and wide, a complete water-borne system of sewerage was designed by the Writer in 1926-1928 and the execution of this scheme is now nearing completion. It involves the installation of Power-driven Pumps and up-to-date Sewage Disposal works at a very heavy cost.

At Montgomery in Punjab, all the drains, including the outfall, were designed to carry a flow of 1/20th rain fall per hour, in cunette section, from the commanded area. Rainfall per annum is 10 inches on average. Potable water supply for domestic purposes is 10 Gallons per head per day of 24 hours. At the end of the outfall, balancing storage tanks have been built to store sullage when pumps are not working. Chokeless centrifugal pumps with horizontal spindles, driven by oil engines, lift sullage for 3 to 4 hours in the morning and also in the evening. Overall

efficiency of Pumps and Engines is 40 per cent. Canal water at the rate of 2½ gallons per head per day is stored in open tanks and pumped to different parts of the city to flush the drains. 5000 Gallons of sullage are distributed on sullage Farm, one acre, in 24 hours, 1/3rd of the area of the Farm is to lie fallow every year.

Sewage from New Delhi was carried to Kilokree through outfall Sewer 84″ in diameter which was provided with an overflow for storm water at a suitable point, the diameter of the sewer being reduced from the overflow point onwards to the Pumping Station. At Kilokree, Pumping Station was built to house boilers and three sets of Sewage Pumps (Worthington & Simpsons). Pump Horse Power of each Pump = 146 : Steam per P.H.P. per hour = 16 lbs., 5 lbs. of steam are produced by one lb. of rubble coal 2″ in diameter, which weighs 56 lbs. per cubic foot as arranged in a stack.

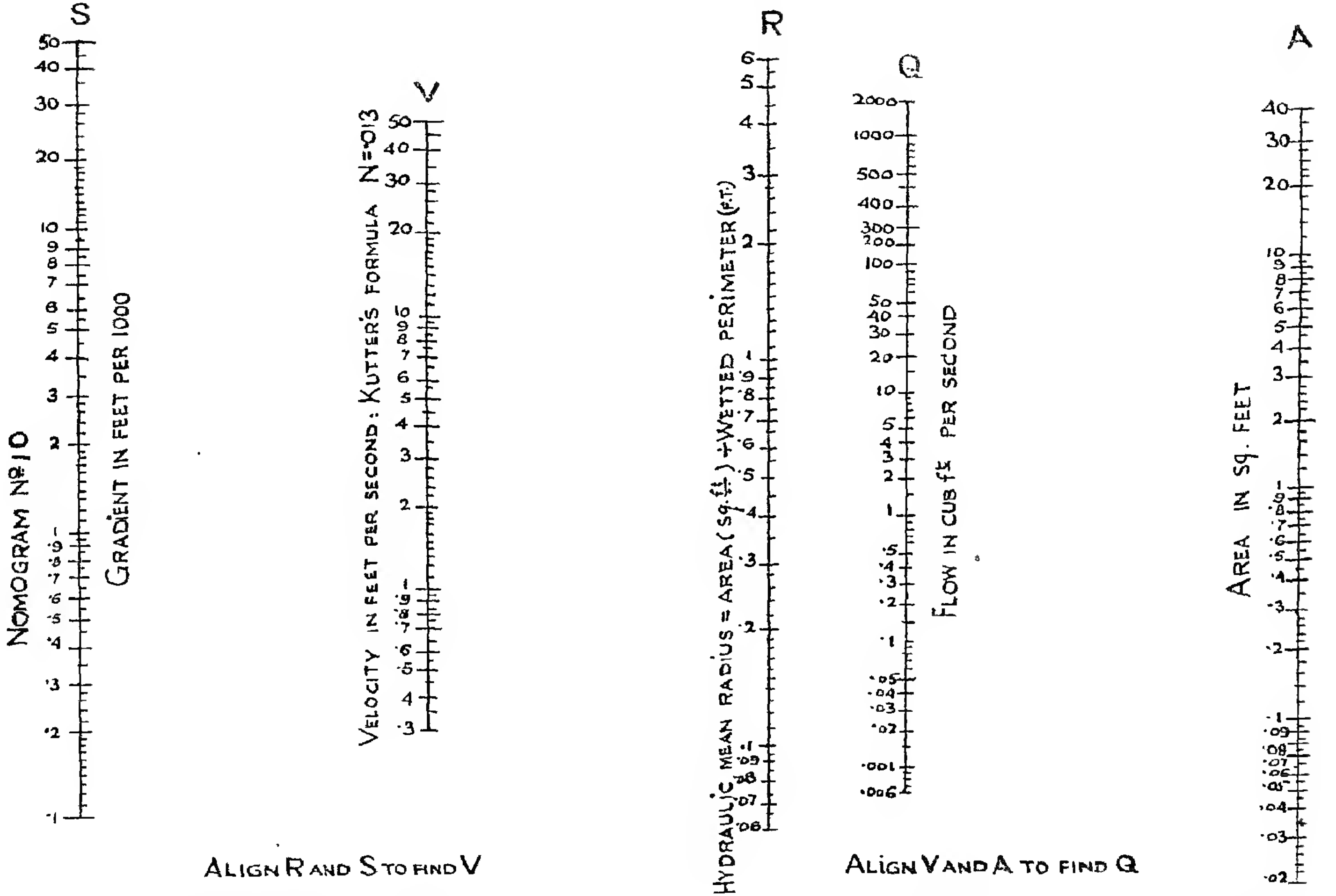
Each set of Pumps consists of 4 vertical plungers driven by reciprocating Horizontal bars actuated from Triple expansion steam engines (in duplicate), fed from steam boilers, for which pure water was used to produce the steam. Pure water was also required for Pump glands' water-seals at the rate of 30 gallons per minute for each set of Pumps. This water was obtained by Pumping Sewage in a high level settling Tank and thence spread through sprayers on aerating filters and thence passed through slow sand filters, the rate of filtration being 36 gallons per square foot per day of 24 hours, on the top surface of the filter: Purified Sewage was also used for condensing water and cooling water for the steam engines. This Purified sewage as it comes out of the Aerating Trickling filters, is stored in large cooling open tanks from which the cooled sewage flows to the condensers and is pumped back to the cooling tanks.

113 D :—*Sewage disposal* :—At Kilokree for New Delhi, Raw Sewage was distributed on the Sewage Farm 1600 acres (400 acres being occupied by buildings, Tanks, filters, etc.), out of 1200 acres, 400 were to lie fallow, the balance of 800 acres was brought under cultivation in 1925. 5000 Gallons of Sewage were distributed on one acre of land in one day of 24 hours and the whole farm was leased to contractors for cultivation who paid Rs. 50,000 per annum. In a few years, the raw sewage rendered the whole farm sick and water logged. The New Delhi Government built a new outfall sewer which carried Sewage by gravity from Kilokree for about 2 miles, further to the south, for treatment by Simplex Plant and then disposal in rain water drains and use by farmers for irrigating fields, watering trees, etc. This Simplex Plant has cost several millions of rupees.

Some of the Sewage in Bombay is disposed of by Simplex Plant at Dadar Ry. Station at a very high cost. Lately Mr. N. V. Modak, an eminent Engineer, has installed a special Plant to treat the Sewage at a very reasonable cost: (Refer to Bombay Municipality).

The Writer put up in 1937 an installation at the New T.B. Hospital at Kasouli near Simla. Advantage of the sloping hills was taken to avoid Pumps. The Sewage was carried to a digestion tank to store 10 gallons of Sewage per head per day of 24 hours; thence to sprayers to aerate the effluent on the top of an aerating filter, thence to a slow sand filter and finally to Sewage Farm, all by gravity. This arrangement has been adopted on his own initiative by Mr. N. V. Modak in Bombay with improvements, *viz.* the slow sand filter has been eliminated and compressed air is forced in the aerating filter instead of using the sprayers. Under this device, a great deal of effluent is aerated and purified at a reasonable cost (The Institution of Engineers (India), Bombay Centre. 27th Annual Report 1947-48).

Fig. 103 A



V as found above is for stone-ware drains: for culverts, rubble drains, etc, multiply V by $\frac{3}{4}$: for earth-drains multiply by $\frac{1}{2}$

CHAPTER XVI.

Hydraulic Observations.

114. (*a*) *Preliminary.*

An engineer is often required to calculate discharges of water in natural storm water drains, in canals, pipes, sewers and rivers. When these channels are dry or carry little water in dry season, the best thing is to select a straight run of the channel with more or less regular section and regular bed slope. The 'straight run' should be as long as possible upto 2,000 feet. Take cross-sections at convenient places and the longitudinal section of the bed. By Kutter's formula, the discharges can be worked out to any required depth above the bed level.

If the channel carries a large quantity of water, then the following methods are to be used to calculate the discharge.

(*b*) *Soundings.*

To plot the cross section, soundings are taken as described below.

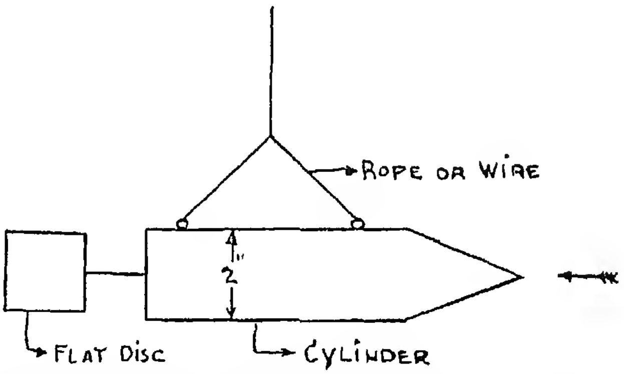
In small streams with depth of water about 10 feet, it is usual to stretch a cord from bank to bank and to determine the depth at equal intervals along the cord. The mean of three such cross sections at intervals of say 50 or 100 feet along the course of the stream is taken as average cross section. The depths of water at stated intervals are measured by a sounding pole, divided into feet and tenths and provided with a flat disc at its lower end to prevent its sinking into the soft parts of the bottom.

If the depth of the water is more than 10 feet, boats or steam launches are used for taking soundings, and the measuring rod is replaced by a sounding line of hemp rope, preferably with a copper core.

If the channel is a deep and broad river with a swift current, boats or steam launches with a sounding line of steel wire, weighted at bottom with a heavy weight, are used.

The weight should preferably be of the form shown in fig. 103 conical at one end and fitted with a disc at the other to keep it in position.

Fig. 103 E

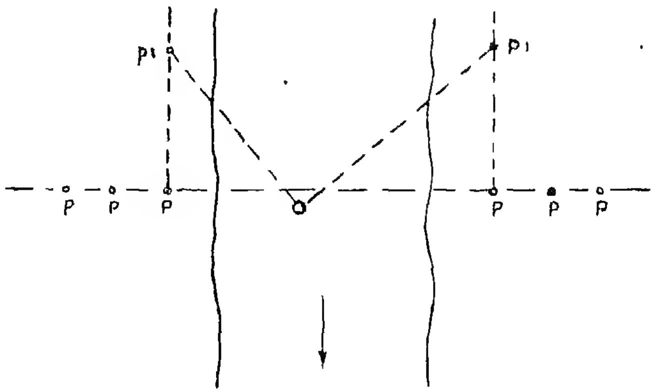


Mr. F. F. Haigh, C.I.E., once Chief Engineer, Punjab, has invented a 'Depth-Meter' to measure the depths of flow in deep turbulent and swift rivers. It is based on Boyle's law, *viz.* PV = Constant. The observations so far made give constant results. The inventor is perfecting it. It is a very ingenious device.

Kelvin tubes have also been tried in Sindh with not accurate results. While gauging the discharge of river Sutlej at Sunni, about 20 miles from Simla, in 1909, the writer used fine steel wire and a heavy weight of stone for soundings between the two vertical cliffs which form the two banks of the river, which passes through a gorge with a velocity of 10 to 15 feet per second, precluding use of a boat or a launch.

In the fig. 104, P P P.......... shows a line of flags fixed on both banks of a large river and at right angles to the centre line of the river. The poles P¹ and P¹ also show flags fixed on the banks up-stream of the line P P P...... The lines P¹ P¹ are at right angles to the line P P P ....

Fig. 104



An observer coming down the river in a boat can by means of a pocket sextant, read the angles P O P^1 and P^1 O P^1 when the boat drifts down to the point O. A steam launch will serve this purpose better. By this method the position of the point O can be fixed along the line PPP.........................It is granted that the position of the point O cannot be measured by a tape or chain from the bank of the river.

Two surveyors with theodolites at points P^1 P^1 can also fix the point O.

115. *Measurement of velocity.*

When the velocity is observed at one or more points in the cross section of a stream, the process is termed " Point measurement ". When the mean velocity on a line in the plane of cross section is found directly, it is known as integrated measurement. Velocity measuring instruments are of two kinds namely (*i*) floats and (*ii*) fixed instruments. Fixed instruments give the velocities in one cross section of the stream only. Floats give the average velocity in the " run " or length over which they are timed and not that at one cross section. Floats are used only in open streams but fixed instruments in open streams and some times in pipes.

With most instruments, time observations are necessary. For this purpose, chronometers, stop watches and sometimes ordinary pendulum are required.

(*a*) Floats include surface floats, sub-surface floats, and rod floats.

(*i*) *Surface floats* :—Divide the cross section of the stream in 10 or more parts. Take the surface velocity in the middle of each part.

Surface floats are used for point measurement. For surface velocities, surface floats alone are, in regular stream, the best instruments unless there is considerable wind. To guard against wind disturbance, globular floats like the shape of an orange are recommended. For deep water, empty bottles will serve the purpose. For shallow water, a circular disc with a painted knob will do well.

Select a straight " run " of the channel over 100 feet in length ; mark the ends of this " run " by poles. Take tho float in a boat a little upstream of the actual point and place the float in water. When the float drifts down to the actual point, mark the time by a stop watch. When the float reaches the end of the ' run ' mark the time. This will give you the surface velocity. Calculation of mean velocity from the surface velocity is treated below.

Care must be taken to see that the float travels down more or less in a straight line parallel to the bank and does not deviate too much from it. This method is only approximate but is very rapid ; and when systematically checked by more accurate methods, is very useful. Take several such surface velocities and find out the mean of all.

To work out the discharge of the particular portion of the cross section of the stream, the mean surface velocity at the middle of this part having been found, it is now necessary to calculate the mean velocity along the vertical depth of the stream.

If V_s = the mean surface velocity.

V_m = the mean velocity along the vertical line, then

$V_m = P\,V_s$ ....103.

P = 0·79 to 0·91 : 0·85 for river Danube.

(P. I. C. E., Vol. 91, page 399), where P is greatest for sandy beds and lowest in the case of rivers carrying gravel of fist size.

P = 0·835 for Switzerland mountain streams with stony beds.

= 0·93 for Punjab rivers with extremely fine sand, carrying large volumes of water.

= 0·90 do. do. do. when in flood in hills.

= 0·90 to 0·95 for American rivers in floods.

= 0·89 in Punjab as deduced by Mr. Goodman, (excluding hill streams).

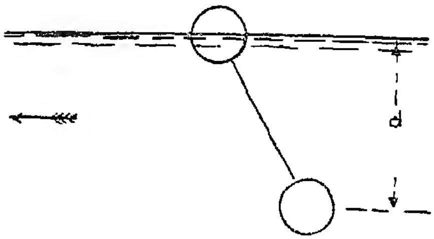
The general law is fairly well known that P increases as the depth increases and as N, the *rugosity factor* in Kutter's formula decreases. In small and rough channels, P has been found as low as 0·60. Bellasis in his book page 169 gives, that the value of P varies from 0·83 to 0·95 as the depth increases and the smoothness of the channel increases, in other words the value of N decreases.

Parker in his book ' Control of water ' page 62 gives a table showing that the value of P decreases as the ratio $\frac{\text{width of stream}}{\text{depth}}$ increases. The reader can assume the value of P from the above considerations and work out the mean velocity on the vertical and then the discharge of the portion of the cross section of the stream in question.

The mean velocity on a vertical occurs at a point 0·6 of the full depth of flow from the bed. (Hill streams excluded). This method is approximate but useful when rivers are in high floods and more accurate methods are not physically possible.

(*ii*) *Sub-surface floats* :—A float used for measuring the velocity at a given depth below the surface is called a 'double float'. A submerged 'Lower float' somewhat heavier than water is suspended by a thin wire from a buoy which moves on the surface, as shown in the figure 105.

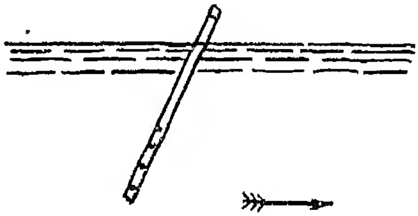
Fig. 105



The buoy or top float is usually a hollow steel ball so is the lower float which is made a little heavier than water.

The depth 'd' of the lower float is regulated by the length of the cord. The velocity is then obtained by timing the top float over the known distance. This is the average velocity for the depth d. Bellasis recommends the use of a multiple float with several equidistant submerged floats, the lower ones heavy and the upper ones light, the distance of the lowest from the bed and of the highest from the surface being half the distance from the two adjoining floats. All these floats are best suited for slow currents and where the river or channel is free from weeds.

Fig. 106



(*iii*) *Rod floats* :—Having taken the cross section of the stream by soundings and having plotted the same on paper, one can see the depths of flow in the stream in different parts of the cross section. Gauge the velocity of each part separately.

A rod float is a cylinder of tin sheet, ballasted with shot, so that it floats upright in still water. In flowing water it tilts, as shown in fig. 106, because of the differences in the velocities of the stream, and it moves unsteadily owing to the irregular movements of water. Its length should be such that the bottom end does not get stuck in weeds or in mud or in sand.

Hollow rods of sheet iron are far superior to those of wood and should always be used. The rods are made in short lengths, one screwed on to the other to make up the required length and loaded with shot in the bottom part to keep the float vertical.

Select a length of over 100 feet where the channel is straight and uniform in section. Mark three points 50 feet apart by flag lines across the channel. Stretch wires from one bank of the channel to the other with pendants fixed at points 10 feet apart. Now move the float from one pendant to the corresponding pendants on the 2nd and 3rd cross wires, parallel more or less to the main flow line of the stream. Time the journey of each float by means of a stop-watch. Thus we get the velocity of the rod float at each pendant, which is supposed to be in the middle of each part, the cross section having been divided into suitable parts.

The velocity of the float so obtained, is the average velocity on the vertical plane in which the float moves.

The velocity near the bottom of the channel is generally slow and for this reason it is probable that a rod reaching too close to the bed would move too slowly and this would affect the average velocity. Accordingly Francis, a celebrated French hydraulician, recommends that the length of the rod should not be more than 0·99 of the full depth of flow along the line on which the rod is intended to move. If the length is 0·93 of the full depth, the velocity found is 2 per cent in excess of the real.

In rough earthen channels, a rod of the proper length can hardly ever be used but allowance can be made for its shortness; for detailed information see "Roorkee Hydraulic experiments, Cunningham. Roorkee 1881."

Francis states that the real mean velocity of the float depends upon the width and the depth of the channel, roughness of the bed and the sides of the channel, and the clearance between the lower end of the rod and the bottom of the channel. The actual velocity varies from 0·96 to 1·00 of the observed velocity.

The method of gauging by rods is accurate only upto 2,000 cusecs discharge in channels, above that figure, current meters are recommended to gauge the velocities. (Fig. 107).

The floating rod method is certainly far superior to the surface float method of gauging. The rod float gives integrated measurement on a vertical line. It is not suitable for large rivers carrying turbulent flow. A rod float should be used five times on the same length of the stream and average taken.

Mr. S. L. Malhotra, I.S.E., in his paper No. 272 presented before the Punjab Engineering Congress (volume 32—1945) describes rod floats in a proper manner.

Fig. 107

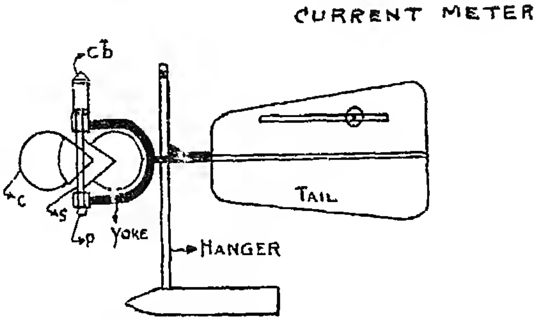
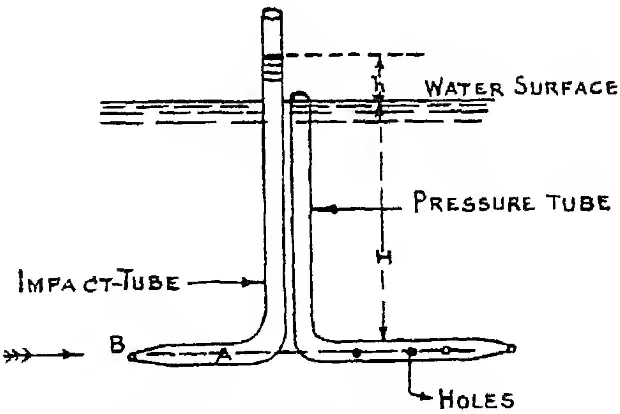


Fig. 107 shows a current meter. The same are of different makes, each maker claiming special merits of his make : P.W.D. should be consulted about the best make and instructions as to its use are given by the makers.

(b) Fixed Instruments.

Fig. 108

(i) *Pitot's Tube* :—This in its simplest form as invented by Pitot consists of a glass tube with the lower end bent through 90° (Fig. 108).



It is used to measure the velocity head of a flowing liquid. It is placed in the moving liquid with the bend pointing upstream. The liquid flows up the tube until all its kinetic energy is converted to potential energy ; the velocity of the fluid may then be estimated by the height of the liqnid in the tube.

Applying Bernoulli's equation to the points A and B one inside the tube and the other outside.

Total energy at A = total energy at B

$$H + h = H + \frac{V^2}{2g}$$

$$h = \frac{V^2}{2g}$$

It is difficult to read the height h, specially when the water surface is turbulent. Mr. Darcy added a 2nd tube, whose lower bend points down stream and this is called pressure tube. As the water passes down by the orifice of this 2nd tube, it creates a little vacuum in this tube and the water level inside the tube falls below the water surface of the stream through a height nearly equal to h. To guard against this Darcy made holes at right angles to the lines of flow in the sides of the lower bend of this tube. This keeps the water surface inside this tube nearly at the same level as the water surface outside. Thus the velocity head is the difference of water levels in the two tubes.

Thus theoretically, in the first tube water stands at a height equal to the static pressure plus the dynamic pressure; while in the second tube the static pressure alone is indicated.

In well made instruments, the lower bends are made of copper fitted with stop cocks. The top parts are made of glass. The 2 top ends are joined together and fitted with an air-cock which can be connected to a rubber tube, through which air can be sucked out a little. This arrangement enables the columns of water in the 2 tubes to move up, the difference of level remaining the same. Thus the difference of level between the water surfaces inside the tube can be read more conveniently by a graduated scale.

Theoretically, if h be the difference in level of the water columns, then $V^2 = 2\,gh$, gives the velocity of the current. In actual practice, it is extremely difficult to prevent the pressure holes in the bend of the 2nd tube on the down stream side, from being exposed to some action by the current in the nature of suction, producing a depression of the corresponding water column. Hence as a rule $V = C\sqrt{2gh}$, where c is a coefficient of the instrument.

The manufacturers of this instrument so devise the apparatus as to keep c as unity.

The instrument can be rated by moving it several times in still water with known velocities and the velocity head 'h' can be read. From known velocities and known readings of 'h' the value of c can be found from the formula $V = C\sqrt{2gh}$. The several values of c so found can be averaged to arrive at a 'Constant' of the instrument in hand.

The instrument can be inserted through a stuffing box, into a pipe carrying known discharge with a known velocity. The readings of 'h' can be taken and the coefficient c can be ascertained.

This instrument is manufactured in several types by different makers, each claiming special merits for his apparatus.

A Pitot's tube is generally used to ascertain velocities in pipes and in small very smooth channels. It is also very useful to ascertain the velocities of jets on impulse wheels. In this case the impact tube is connected to a mercury column or the ordinary Bourdon pressure gauge.

It is well adapted for observations in channels with depth of flow from 5 to 6 feet. It has the advantage of requiring no time observations. It is now so well manufactured that it can be fixed to rod and lowered to ascertain velocities in channels carrying silt laden water, specially near the border and the bed, where a current meter can not be used for fear of its mechanism being clogged by the silt in water.

The instrument can not be well used for very low velocities. See P.E.C. paper No. 265, Vol. 32 (1944).

At hydraulic research station, Malikpur in Punjab, the following Pitot's tubes are in use.

Prandtl type for general work. The Gallen kemp Industrial type for measuring high velocities.

Universal type for recording flow in different directions.

Benzl tubes are used for measuring large velocities.

This instrument is also very useful to determine the direction of maximum velocity.

(*ii*) Miscellaneous Pressure Instruments :—

Ingenious hydraulicians have invented other pressure instruments which are seldom used for gauging velocities. Only mention of some of these is made here. They are

Piezometers.

Porrodil's Hydrodynamometer.

Hydrometric Pendulum.

Bruning's Tachometer.

Further information can be obtained about these from firms dealing in such instruments.

116. *Water levels and surface slopes.*

(*a*) It is often necessary to observe water surface levels in canals, rivers, mountain streams, lakes and tanks by means of gauges. A gauge in its simplest form is a vertical scale fixed in a stream and graduated to lengths of a foot like a levelling staff. Sometimes one sees a gauge marked on a pier of a bridge built across a river, the zero being at the average bed level so that the reading gives the depth of water. Sometimes the gauge is a plate of enamelled iron, screwed to a wooden post fixed in the bed or spiked to an abutment of a bridge or culvert.

If the gauge is exposed to current, it may be damaged by floating bodies.

To overcome this difficulty, the gauge is often fixed in a small masonry well built in the bank of the stream and connected with it by means of a small channel.

The above class of gauge is not of very great accuracy. Therefore floating gauges have been invented. The graduated rod or staff is attached to a float at its lower end which rises or falls with the water-level. The rod travels vertically between guides and it is read by means of a fixed pointer on a level with the eye of an observer. The float and rod should be of metal so that they may not absorb moisture, the float being conical.

Where a gauge does not exist, the water level can be measured from a fixed point, such as top of parapet of a bridge, whose level with reference to the bed of the channel being known.

On huge artificial lakes, self registering gauges are seen working. A pencil actuated by a float moves vertically and records on a band of paper, wound round a drum, moving horizontally by clock work.

The level of still water can be observed with great accuracy by Boyden's Hook-gauge, which consists of a graduated rod with a hook at its lower end. The rod slides in a vertical frame carrying a fixed vernier and is worked by a slow motion screw. The rod is moved up and down till the point of the hook touches the water surface and the pointer on the rod above gives the reading by means of a vernier. This kind of gauge is not of much use in rivers where the water surface oscillates.

A pointed plumb-bob, hung over the water by a steel tape from the top of a masoury wall or a pier or a bridge parapet gives an accurate reading. The point of the bob touches the water surface and a vernier can be used to read the decimal parts of the tape.

(*b*) Surface slopes.

The longitudinal section of a stream can be taken by means of a level and staves if the depth of flow is under five feet and the current is not very swift. If the depth exceeds 5 feet, then soundings are taken, the sounding line being provided with a flat disc at bottom. Thus the longitudinal section of the bed of the stream is taken and plotted.

The slope of the water surface is easily measured by means of a level and levelling staves. To guard against the inaccuracies caused by the waves in the stream, two ditches are dug from the extremities of the slope length, both leading into gauge wells. The water surface in these wells is observed by means of a level and levelling staves. The distance between the two wells gives the length of the stream and the difference between the water levels gives the surface slope.

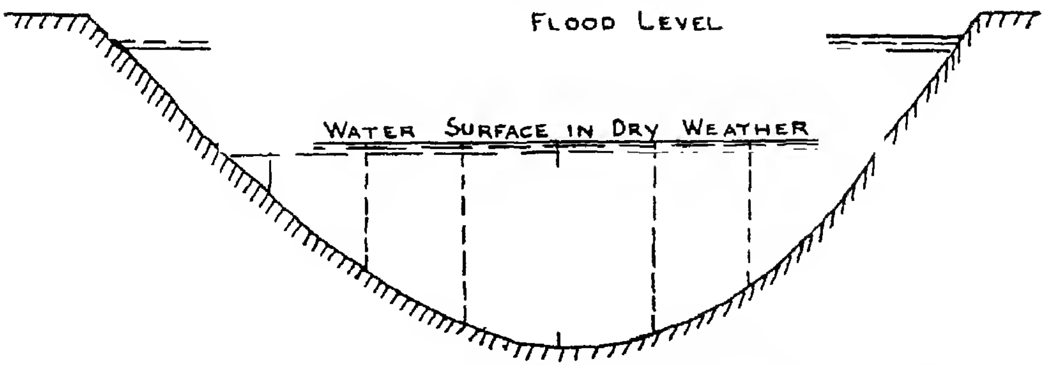
117. *Discharges.*

(*a*) Discharge from an orifice can be ascertained by means of filling a large vessel of known capacity and timing the operation.

For Simla water supply scheme, the writer put up v notches across nallas carrying spring water under one cusecs. For 5 to 10 cusecs sharp edged weirs of wrought iron plates fixed in masoury walls were used. For discharges upto 25 cusecs, a channel about 10 feet wide and one foot deep was made in the bed of the Nauti-khad near Simla. About 100 feet length of the stream was taken as a straight run : cross sections of this temporary channel were taken at 10 places, 10 feet apart with a level and staves. Velocity was calculated from Kutter's formula, with N as 0·35, the bed of the stream consisting of pebbles and coarse sand, the sides of the temporary channel of rubble stone walls.

(*b*) If the stream is a river of considerable depth, three or more cross sections are taken at points where the river has a well defined run and well defined widths.

Fig. 109



The cross sections are plotted and each section is divided into suitable segments. Average velocity in each segment is observed. Area × mean velocity of each segment gives the discharge of that segment. Sum of discharges of these segments gives the discharge of the river at the particular cross section. Average of such three discharges is the discharge of the river. (P. E. C., Vol. XXXIII, 1945).

When the river is in flood, the flood level is to be noted. When dry, the flood marks are to be ascertained and their levels taken.

If A be the area of cross section of the river flow in dry weather, r the hydraulic mean radius, Q the discharge and when in flood, these be, A^1, r^1, Q^1. Then $\frac{Q^1}{Q} = \frac{C\, A^1\, r^1}{C\, A\, r}$ ....104.

The bed slope of the river may be scoured a little during floods; but this will not affect the flood discharge appreciably

Sometimes discharges of a river are ascertained by means of a gauge, a device which reads the elevation of the water surface in a stream with respect to a certain datum. It either reads in feet, with zero at the lowest bed level of the river or directly in reduced levels.

The discharges of the river are observed and worked out at different depths of flow by taking cross sections at well defined points and by Kutter's formula. A discharge curve is plotted on a piece of paper and the relation between the gauge readings and the corresponding discharges of the river is established from the curve: Thus a gauge reader notes the gauge readings daily and informs his superior about it. When we read in newspapers that the gauge reading at Attock on Indus River is getting higher, we know the river is rising by so much and the discharge is increasing.

The discharges of channels are measured by current meter (Fig. 107) if velocity is more than 0·75 ft. per second, by rods if v is less than 0·75 but depth more than 1·0 ft. and by surface floats using coefficient 0·7, if velocity is less than 0·75 ft. and depth less than 1·0 ft.

(c) *Flume*:—Discharge observations in an open stream are greatly facilitated by the construction of a 'flume'. A short length of the channel is constructed of masonry or timber. The sides may be sloping but are preferably vertical. In the absence of silt deposit, the section of the stream is known from the water level, and if rod floats are used, they are of one length. The surface velocities are observed at different points and a mean is worked out. The discharge is thus found.

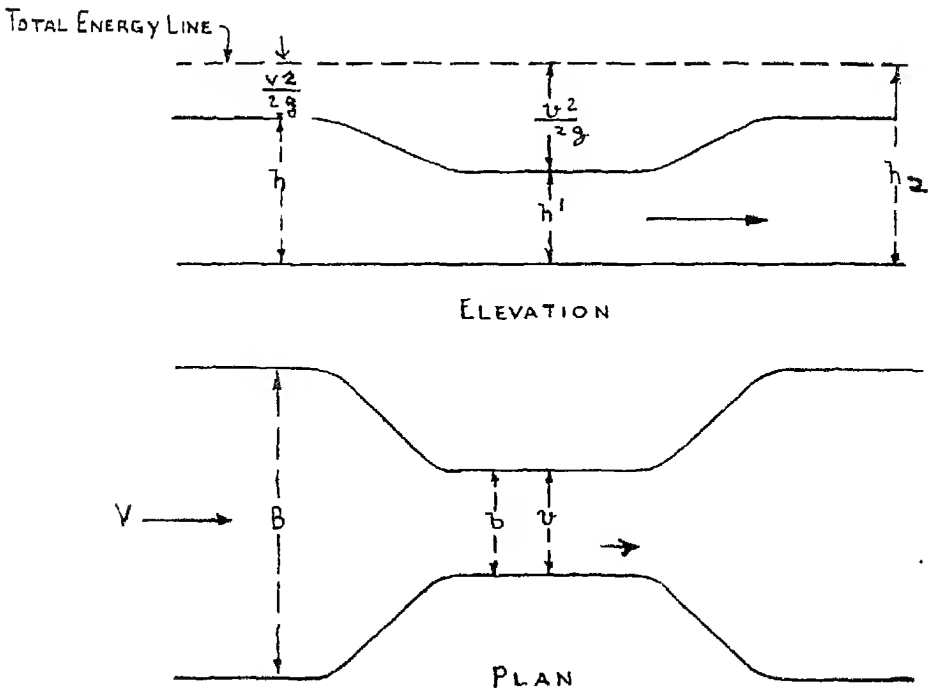
(The Lea Recorder Co., Ltd., Manchester, have constructed recorders to record the flow through channels by means of introducing standing wave flume in the length of the channel. The difference of levels in the water surfaces of the main channel and the throat of the flume are indicated by floats and recorded on moving drums by clock-work and pencil pointers).

The surface slope and bed slope can be ascertained by means of a level and two staves. Velocity can be found by Kutter's formula. Thus the discharge is worked out.

(d) *Venturi flume in a channel.*—This is also of importance and is therefore given here.

The quantity of water flowing along a channel can be measured by restricting the width as shown in fig. 110. This is known as venturi flume and corresponds to the throat of a venturi meter used for the measurement of pipe flow.

Fig. 110



The quantity of flow past the normal section and that past the narrow section are the same.

$$Q = B\,h\,V = b\,h^1\,v.$$

Evidently v is greater than V. The water level in the narrow portion falls. The total energy remains practically constant as shown in the elevation. By Bernoulli's equation, the total energy $h_2 = h_1 + \frac{v^2}{2g} = h + \frac{V^2}{2g}$.

The discharge $Q = k\,(h_2\,h_1{}^2 - h_1{}^3)^{\frac{1}{2}}$ ....105.

Where k is a constant for a given flume, which includes all losses.

Q is maximum when $h_1 = \frac{2}{3}\,h_2$ ....106.

Q (maximum) $= 3{\cdot}09\ b\ h_2^{\frac{3}{2}}$ ....107.

$$Q = \frac{Bh \times bh_1 \sqrt{2g}}{(B^2h^2 - b^2\,h_1{}^2)^{\frac{1}{2}}}\ (h - h_1)^{\frac{1}{2}} \quad \text{..........108.}$$

This should be multiplied by a coefficient, found experimentally, in order to allow for any losses in the flume. Generally this coefficient is approximately unity.

It is possible for a hydraulic jump to occur in the down stream portion of the flume if the conditions are favourable.

See article 96.

Venturi flumes have been used at the heads of water-courses taking off from canal distributaries, vide paper No. 125 P. E. C. (1929) and paper No. 205 P. E. Congress (1937).

(e) *Discharge through pipes.*

If a pipe has been in use for some years, it gets incrusted on the inside. Its clear diameter is measured by means of a rod with a hook inserted through a stuffing box. For obtaining the mean diameter in a length of the pipe, one method is to fill the pipe with water by means of a 5 gallons measure; count the number of gallons poured in and find out the cubic contents of the water. The diameter of the pipe can thus be ascertained.

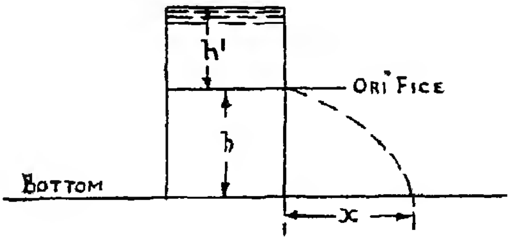
The discharge through a pipe can be ascertained by reading hydraulic pressure gauges at two or more points at a known distance apart. The difference in pressures divided by the distance gives the hydraulic grade which gives the discharge.

Venturi meters are nowadays used to record discharges through pipes.

(f) *To find coefficient of velocity for discharge through an orifice.*

When the water rushes out of an orifice, the jet describes a parabola. The velocity of the jet is found by observing its range on a horizontal plane.

Fig. III



$X = 2\,(h\,h_1)^{\frac{1}{2}}$ ....109.

$V = c\sqrt{2gh_1}$. Thus when x is measured, h is measured, and h_1 is measured, the value of C can be ascertained.

(g) A weir in a thin wall affords the best means of measuring the moderate quantities of water. For discharges over weirs and sluice gates in Barrages across the rivers. See paper No. 272 P. E. Congress (1945).

The method consists in measuring the depth of water over the crest of a weir, including the velocity of approach and the use of this depth or head in a suitable weir formula. Rectangular weirs of two types *viz.* sharp crested and broad crested are used for discharge observations and have the following characteristics :—

(*i*) *Sharp crested weirs.*

The discharge over a sharp crested weir is effected by (*a*) the sharpness of the crest, (*b*) smoothness of upstream face of the weir, (*c*) distribution of velocities in approach channel, and (*d*) ventilation beneath the Nappe.

The discharge is quite accurately given by Rehbock's formula.

$$Q = \frac{2}{3}\ \sqrt{2g}\ \ L H^{\frac{3}{2}} \left(0{\cdot}605 + \frac{1}{320\,H-3} + \frac{0{\cdot}08}{P}\ H \right)$$

where L = length of weir and P = height of crest above floor of upstream approach channel.

It is, however, difficult to maintain a weir in the standard condition and a departure from such conditions means increased discharge. This type of weir is used mainly for experimental purposes and is raroly found on the irrigation channels.

(*ii*) *Broad crested weirs* :—A broad crested weir, working modularly, forms accurate means of measuring the discharge and this method is in use on most of the Punjab canals now. The previous practice, which is being replaced, was to frame a discharge table for the earthen section of a channel, from accurately observed discharges in the form $Q = K\,D^{\frac{5}{3}}$. The discharge over a broad crested weir is given by the formula $Q = C\,L\,(h + ha - hf)^N$ where $C = 3{\cdot}087$, the maximum theoretical value under any conditions and $n = 1{\cdot}5$. The head due to friction hf varies with velocity, silt contents, water temperature and viscosity, and is negligible. Further, due to local peculiarities of construction the theoretical values of C and n are not attained in actual practice and the weirs are calibrated before being applied for discharge measurements. The process of calibration consists in actual observations of discharges with extreme accuracy and care at $\frac{1}{4}$, $\frac{1}{2}$, $\frac{3}{4}$ and full supply conditions and thus at least seven observations are collected at different gauges. The equation for the meter is then worked out by the method of least squares applied to logarithms of the observed data. The head to be used in the formula is the depth of still pond level over the crest, *i.e.*, the gauge plus head due to the velocity of approach. It is, however, more convenient to find the relation of the discharge with direct gauge reading without adding the head due to velocity of approach and this shall apply only within the range of gauges observed. Two meters have recently been calibrated and the following relations have been obtained :—

Site of meter	Equation in terms of H = h + ha	Equation in terms of Gauge readings
R. D. 27000 L.B.D.C.	$q = 3{\cdot}186\ H^{1.503}$	$Q = 3{\cdot}112\ G^{1.54}$
R. D. 1000 L. J. C.	$q = 3{\cdot}033\ H^{1.503}$	$Q = 2{\cdot}968\ G^{1.582}$

It will be seen that the co-efficient and index of head vary appreciably from the theoretical values, and the reasons can be traced to peculiarities in construction. It is held by Parker that "a weir is a very accurate measuring instrument but like all other measuring instruments it must be carefully standardised in order to get the best result from its use and the standard should be copied in details, which at first site appear to have no real effect upon the discharge." The necessity for standardisation of such weirs in respect of shape and splay of wing walls, position of gauge wells, etc. is manifest from the above facts.

The accuracy of calibration depends upon the accurate measurement of Q and H, the two elements in the weir formula, and precautions like proper selection of discharge site and siting of gauge, upstream of draw down point with zero at mean crest level and calculation of velocity of approach at gauge section line, should be observed.

Observation of discharges should be taken with a pair of newly rated meters to obviate effects of temperature. Two observers should simultaneously take the observations in the form of a double traverse first from left to right and then right to left. The gauge should be kept steady during observations and should be read every fifteen minutes. Observations with a variation of ·02 ft. should be rejected.

As stated above, the deviations in the values of C and n from the theoretical, shall depict the effect of local peculiarities in the construction of the meters, the theoretical values being for ideal conditions.

Weir method being the most accurate one, especially for smaller discharges, is used for experimental and research work. In the Irrigation Research Institute at Lahore, the contrivances used are round crested weir, sharp crested weir and V notches. The weir have been calibrated by actual volumetic measurements and calibration curves have been prepared, which for the sharp crested weir agrees remarkably closely with Rehbock's formula. The V notches are made in steel plates with brass sharp edges. A 90° V notch was calibrated and gave the formula $Q = 2{\cdot}25\ H^{2\cdot5}$ with H from $0{\cdot}1$ to $0{\cdot}6$ against the following formulæ arrived at previously :—

Thomson	$Q = 2{\cdot}54\ H^{2\cdot5}$	with H from $0{\cdot}1$ to $0{\cdot}6$
King	$Q = 2{\cdot}52\ H^{2\cdot47}$	with notch in Comm^{l}. Steel plate.
Barr	$Q = 2{\cdot}48\ H^{2\cdot48}$	,, ,, ,, Polished Brass plate.

No calibration for a 60° notch has been done so far in the Institute. This notch is used for small discharges as the smaller angle gives greater head for the same discharge and more accuracy. In calibrating V notches the gauges were read up to ·0004′ with a vernier scale. In the Institute, discharge up to 5·5 Cs. are measured with sharp crested weirs and 2·0 Cs. with the V notches.

Further investigations regarding discharge measurements.—The subject of accuracy in measurement of discharges is still on the agenda of the Central Board of Irrigation and attempts at research in this direction are being made in various provinces.

In conclusion, the writer of paper No. 272 P. E. C. wishes to stress the importance of accurate measurement of discharges for the irrigation engineers both in practice and research work. The subject is becoming more important with the enhancement of the value of water with the development of irrigation and it is hoped that desired co-operation will be offered by the profession in perfecting the methods being developed by the Discharge Divisions, in different provinces.

118. *Siltometer.*

For a canal engineer it is necessary to know the quantity and the grade of silt in his canal water. The quantity can be ascertained easily by evaporating a known quantity of silt laden water. The grade or diameter of silt particles is found by means of an instrument called Siltometer.

In Punjab, there was in use, in the last quarter of nineteenth century a siltometer, invented by the celebrated Engineer Mr. Kennedy. It is very simple but is said to be incapable of giving scientifically accurate results. (See Punjab irrigation branch paper No. 9.)

Recently the eminent Mathematicians of Lahore Irrigation Research Institute, by name Dr. E. Mckenzie Taylor, D.Sc., F.I.C. ; Dr. V. I. Vaidhyanathan, M.A., D.Sc., I.P. and Dr. N. K. Bose have invented a very good siltometer which is described fully in paper No. 167 of Punjab Engineering Congress, Vol. XX (1933). It is too complicated to be reproduced here.

While taking samples of silt laden water from a canal, it must be noted that the silt charge in a channel varies with depth from top to bottom, being low near the surface and high at the bottom of the channel. Moreover the silt is finer near the surface and coarser near the bed.

119. *Rain gauge.*

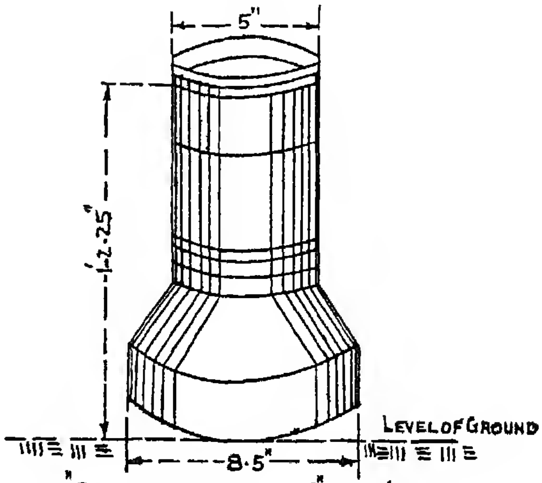
It is sometimes necessary for an Engineer to record rainfall in a district or a particular area ; for this purpose an instrument called a rain gauge is used.

On page 669 of Bombay P. W. D. Handbook, Volume II (1931) description is given of the symons raingauge, prescribed for use in Government departments by the Director of Observatories, Government of India. The gauge is shown in fig. No. 112.

The gauge is built in three parts : (*a*) the base, (*b*) the body, and (*c*) the funnel. Inside the body is placed a glass bottle into which the rain passes through the tube part of the funnel.

To measure the rainfall, take out the bottle or the receiver from the body and pour the water into the measuring glass which is so graduated with marks that each mark corresponds to a certain amount of rainfall on the top funnel. Full information of this subject is available with Meteorological Department.

Fig. 112



5" RAINGAUGE MAX. CAP. 8" OF RAIN.

The above gauges record the rainfall in the interval between two readings taken by the gauge reader. The intensity of rainfall *i.e.*, the rainfall in half an hour or one hour during continuous rain, is not shown by such gauges. Nowadays self recording gauges for this purpose have been invented and are sold in the market. There is an integrating type which records automatically the total rainfall at any time from the beginning of the storm.

CHAPTER XVII.

Miscellaneous.

120. *Viscous resistance of fluids and froude's experiments.*

Fluids in motion are subjected to frictional resistances, mainly due to viscosity, that is, to the resistance to sliding between two adjacent layers of the fluid. It has been found that the motion of a fluid is a steady stream line flow for low velocities only. After a certain velocity is reached, the motion is no longer steady and eddy currents appear. The velocity, at which the steady flow changes to eddy flow, is known as critical velocity.

Froude made investigations to find out the frictional resistances of surfaces moving in water. He built long tanks, filled them with water and moved long wooden boards, whose surfaces were treated with varnish, calico, and sand in turn. The boards were moved with known velocity, from the results of his experiments he concluded :—

(*i*) The frictional resistance varies with the square of the velocity and also with the nature of the surface.

(*ii*) Frictional resistance per square foot of the surface decreases as the length of the board increases, but is constant for long lengths.

Total frictional resistance in lbs. $= f\,A\,V^2$: $f =$ frictional resistance per sq. foot of a given surface at unit velocity in pounds : A = area of wetted surface in sq. feet : V = velocity of surface in ft. per second (for values of see British association reports, 1872-1874, also Encylo -Brit. article Hydro-Mechanics).

Knowing '*f*' for surface of ships, the resistance of a ship to motion can be found, the resistance of the head of the ship to wave motion is also to be added. Ship builders by means of experiments know the value of '*f*' and the value of head resistance and design the Horse-Power of ships accordingly.

Prof. Unwin arranged to revolve a disc at a known speed in a liquid and obtained the coefficient of friction of the disc's surface by measuring the work done. (P.I.C.E., Vol. 80, P. 220).

In the case of ships moving in water against a viscous resistance and wave resistance, the latter due to the formation of surface waves, is a gravity resistance as the wave gains in potential energy. The non-dimensional factor governing the gravity effect is known as Froude number and is the ratio $\frac{V}{\sqrt{(L\,g)}}$,where L is the length of ship. It is also expressed as $\frac{V^2}{g\,L}$ $L =$ length of parameter influencing flow :

Froude number is criterion for turbulent flow in short pipes and open channels, also for shooting flow with high velocity in open channels where V is greater than critical velocity and gravity forces are dominant.

Sir Isaac Newton and Poissenille investigated the phenomenon of viscosity in details and their mathematical treatment of the subject is worth reading by those interested in it :

Coefficient of viscosity of a fluid $\mu = \frac{f}{\theta}$, f being resistance per unit area (= viscous stress) : θ being angle of distorsion due to viscosity : also $\mu = \frac{M}{TL}$: M = unit mass : L = unit space : T = unit time :

The relation between the coefficient of viscosity and the density is known as the kinematic viscosity denoted by K : density D in engineer's units $= \frac{W}{g}$: also taken in Absolute units : $K = \frac{\mu}{D}$: W = weight of one cubic foot of the fluid.

121. *Reynold's experiments.*

Osborne Reynold an eminent physicist made detailed experiments of the flow of water through pipes. He ascertained different discharges, different velocities, different hydraulic grades for pipes of different diameters. He also noted the different temperatures of water from time to time. He found that the flow of water consists of two types. (*i*) For low velocities, the flow is in stream lines. As the velocities increase, a point is reached where the flow is turbulent. The velocity at this point is called critical velocity. He also found that the value of this critical velocity varies inversely with the diameter of the pipe and also inversely with the temperature of water. It also depends upon the density of the liquid and its velocity.

He arrived at the following formula :—

$$\frac{V_c d}{k} = 2{,}000 \qquad \ldots\ldots 110.$$

V_o = lower critical velocity.

d = diameter of pipe.

k = ratio $\frac{\mu}{D}$

μ = coefficient of viscosity.

D = Density of fluid.

all in C. G. S. units.

There are also intermediate critical velocities between the lower critical velocity and the higher critical velocity the flow changes from stream line to turbulent flow gradually.

Reynolds equation for higher critical velocity is $\frac{V_c d}{k} = 2{,}500$ .... 111.

2,000 and 2,500 are called Reynold's numbers. For any liquid in motion, if the number is below 2,000, it is stream line flow. If above 2,500, it is turbulent flow. These values hold for all fluids, at all velocities and temperatures.

Engineers generally deal with problems where the flow in pipes is turbulent and velocities used are above the critical velocity; and Reynolds numbers interest very much those engaged in research work.

Paper No. 263 of Punjab Engineering Congress (1943) describes the results of Osborne Reynolds, in a more detailed way. For complete account of Reynold's experiments see philo. Trans. 1883. Proceedings Royal society Vol. 74: The ratio $\frac{\mu}{D}$ is known as "kinematic viscosity".

Reynolds number is criterion of Dynamic similarity as the viscosity forces are dominant: and Laminar flow as in smooth pipes at very low velocity or seepage through sand when R, Reynolds number is less than 2,500.

Example:—The density of a fluid is 0·8 and its coefficient of viscosity is ·01 in C. G. S. units. Find its lower critical velocity:

From formula 110, $\frac{V_c d}{k} = 2{,}000$:

k = ratio of coefficient of viscosity to Density $= \frac{\mu}{D} = \frac{\cdot 01}{\cdot 8} = \cdot 0125$; $\therefore$ $V_c = \frac{2000 \times \cdot 0125}{d} = \frac{25}{d}$ c. m. per second if d is taken unity.

Example:—Oil weighing 57·2 lbs. per cub. foot at a temperature of 60° F is to be pumped through a C. I. main 6 inches diameter (D) 1000 feet long at the rate of 20 tons per hour: Find the Horse-Power required if the coefficient of friction and roughness and viscosity μ be 0·589 in this case:

20 Tons of oil in one hour = 12·44 lbs. per sec. or $\frac{12 \cdot 44}{57 \cdot 2} = \cdot 218$ cu. ft. per second: velocity $= \frac{\cdot 218}{\text{area of } 6'' \text{ Pipe}} = 1 \cdot 11$ ft. per second: see Formula 52(*a*): Head lost in friction $h = 4\mu L V^2 \times \frac{1}{2gD}$ $= \frac{4 \times \cdot 589 \times 1000 \times 1 \cdot 11^2}{2 \times 32 \times \frac{1}{2}} = 90 \cdot 7$ feet (of oil) H.P. required $= \frac{20 \times 2240 \times 90 \cdot 7}{60 \times 33000} = 2 \cdot 05$.

122. *Boundary layer.*

In canals it is observed that the particles of water, in immediate contact with clayey soil of the sides, stick to the clay and do not move at all. As we proceed from the side to the centre of the canal, the particles show visible signs of movement down the canal. Their velocity increases slowly with the distance from the side bank and we reach a point where the velocity is marked and definite. The thickness of this band of water is called boundary layer and in this layer, the resistance to flow of water due to the physical condition of the sides of the canal is developed.

It is said that Hele-shaw discovered the existence of the boundary layer and Prandtl conceived the idea of a laminated boundary layor, transmitting the fluid resistance to the surface.

When a ship moves in a sea, in which the liquid is more or less stationary, there is a boundary layer between the exposed sides of the ship and sea water.

Mathematicians like Reynolds and others have gone into the details of boundary layer theory; their analyses and results, though very interesting, are not of much importance to civil engineers dealing with the flow of water in canals (sides irregular and full of weeds), pipes (sides incrustated), rivers with bends, hollow sides, and projecting points. They have found chezy formula $V = c\sqrt{r}\ \sqrt{s}$ quite good for their purpose, the value of c, the constant being found by experiment by several authors. Professor Unwin after careful experiments has arrived at formulæ for discharges through pipes with inside surfaces in different conditions see article No. 71.

The boundary layer theory has been developed into very interesting details which are of use, more academical, than practical to civil engineers.

123. *Energy theory of turbulent flow of liquids*

See paper No. 212 of P.E. Congress by T. Blench, I.S.E. (1938).

The chezy equation $V = c\sqrt{r}\ \sqrt{s}$, has always been, and still is the basis of flow formulæ investigation. Before our present knowledge of boundary layers, there seems to have been no appreciation of the apparent difference between smooth boundary and rough boundary turbulence, except by physicists, and the result was "universal" of the Kutter type for c, which are distinguished by dimensional heterogeneity that renders them of little use as dynamical pointers.

Manning's formula is most suggestive and simple, based on one class of data.

$V = \text{Const. } R^{\frac{2}{3}} s^{\frac{1}{2}}$ (article 109 c).

Where 'R' is hydraulic mean depth and 's' is surface slope for a channel and non-dimensional pressure gradient for a pipe.

The influence of Reynolds was to bring the physicist's investigations on to dimensional lines; but it did not affect engineer's formula noticeably.

Recently some authors have made investigations on the subject without success as to final flow formulæ.

Prandtl talks of mixing velocities. See Forschungsheft V. D. 1361 (Berlin). He imagines the frictional resistance of fluids to arise from transference of momentum between layers by the passage of particles. It is generally observed that in turbulent flow, the particles travel from one side to another, from bottom to top, *vice versa*.

His formula for rough pipes is :—

$$\frac{1}{\sqrt{(4f)}} - 2 \log \frac{r}{K} = 1 \cdot 74$$

For smooth pipes :—

$$\frac{1}{\sqrt{(4f)}} = 2 \log (R \sqrt{4f}) - \cdot 8$$

= radius of pipes. K = average height of roughness projections.

$\frac{r}{k}$ = roughness factor : f = frictional coefficient as used in the chezy formula $\left(hf = \frac{4 \, flv^2}{2gd}\right)$

R = Viscous resistance per unit area of wetted surface (Phil. Trans. Vol. 214).

Mr. Blench's formula for rough flow is :—

$$V = C R^{\frac{2}{3}} s^{\frac{1}{2}} \quad \ldots\ldots 112.$$

C = 89 for rock faced masonry in cement.

= 115 clean hard brick well pointed conduits.

= 123 Dressed masonry cement.

= 158 clean neat cement pipes.

When a mass of fluid moves down in a channel, the rate at which gravity works equals rate of transmission of energy at the boundary layer, plus rate of dissipation within the fluid. This is called energy theory of flow. The formula No. 111 is based on this theory.

Lacey's theory takes into consideration canals in alluvial plains, and rivers in spate with the bed material in active condition. These channels are characterised by the fact that they form their own section, shape and slope in their own incoherent transported material (silt). A rigid channel can only adjust its 'R' (Hydraulic mean radius); a rigid pipe its 's' (slope); but a regime channel adjusts its 'R', 's', and $\frac{P}{r}$, p being wetted perimeter. These terms are all uniquely determinable as functions of the discharge Q and the Lacey silt factor f (article No. 109 d). The energy theory of flow has been taken into consideration by Lacey, as detailed in the institution of Civil Engineers' papers 4736 and 4893.

Lacey is now considering over the fact that in a moving mass of water, small masses strike against each other and produce shocks.

124. *A new treatment of the principles of flow.*

In paper No. 263, submitted to the P.E. Congress in 1943, Philip Claxton, once an eminent engineer in the Irrigation Department of Punjab in India (1930) gives a new treatment of the principles of flow and the following is an abstract from the said paper.

(*i*) When water flows in a canal, the particles adhere to the sides and bottom in a thin film. There are protuberances along the whole of the wetted perimeter. When the particles strike against these protuberances, they are deflected towards the main stream, resulting in the formation of eddies which are the chief source of resistance to the flow. During heavy floods or rains floating matter like leaves etc., are seen going on the surface of water along the centre line of the stream, being deflected from the sides.

Some of the eddies that are formed along the bed of the channel, travel upwards and spread out on the surface and some die in the body of the water *en-route*. These form also another source of resistance offered to the flow. On account of the fall in the bed of the canal, the mass of water is acted on by the force of gravity and there is flow, the energy of the moving water is balanced by the equal and opposite resistance (internal) of eddies. Velocity is merely a condition at which the acceleration of gravity is balanced by the said resistances. It is an index of the working of the two forces, equal and opposite, defined by the rate of flow, *i. e.* so many feet per second.

Gilbert and Murphy, experimenters of California, draw attention to the rhythm or wave motion of flow of water. They observe free flow as oscillating motion, each filament developing a wave length or periodicity of flow. To them this seems to be a fundamental law. The writer has noticed in inundation canals and rivers when in flood that water flows in masses of different small lengths, with different velocities due to different resistances on account of varying condition of the surface of the wetted perimeter. Each mass moves as a separate entity, strikes against the other, resulting in the formation of waves, each mass developing a periodicity of flow. Along the sides of the canal, the wave formation is restrained, the filaments of water jostle each other and bring about eddying motion. Thus the energy, created by the acceleration of gravity, is dissipated.

By velocity we mean an average of different velocities of different masses of water of short lengths, over a limited reach of a canal.

(*ii*) We have considered the accelerating forces (F) impelling flow and the opposing equal forces (F_1) due to resistances. We will now consider the work done by these forces. Let us take first forces F. Suppose the bed slope makes an angle 'a' with the horizontal, the unit mass of water denoted by 'm' and the depth of flow by d. Then the work done by the forces F

$= m\ g$ sine 'a' d foot—pounds. At velocity V, the work done by forces $F_1 = C\ m\ V^2$, c being a coefficient depending upon the nature of the perimeter of the canal.

Thus m g sine 'a' d $= c.\ mV^2$.

This is the fundamental law.

All the above notes relate to clear water, flowing into open channel.

125. *Non-dimensional Factors.*

In article 121 a non-dimensional factor, known as Reynolds number, was defined. This factor has an important influence on all problems dealing with the viscous resistance of a fluid. It is merely a ratio. In article No. 129 it is stated that wave resistance of a ship moving in water also depends on a non-dimensional factor known as Froude number.

On applying the principles of dimensional similarity to other problems in fluid mechanics such as weirs, orifices, etc. it is found that they are governed by other non-dimensional factors. This higher mathematics of hydro-mechanics generally do not fall to the lot of a civil Engineer; therefore further treatment of this subject is closed here.

126. *Compressible fluids.*

(*a*) Properties of gases.

As a gas is easily compressible in comparison with water, the variation in its density is considerable, which fact complicates the calculations on gas flow.

If P = Pressure of gas in lbs. sq. ft.

V = Volume in cu. ft.

t = Temperature as measured by a thermometer.

T = Absolute temperature.

= t + 273 degrees C.

= t + 460 degrees F.

d = Density of gas

W = weight in lbs. per cu. ft. of gas

= dg

Then volume 1 lb. of gas $= \frac{1}{w}$ cu. ft.

Let Kp = specific heat at constant pressure in ft. lbs. units.

Kv = Do do at constant volume in ft. lbs. units.

r = ratio of specific heats.

$= \frac{kp}{kv}$

J = Joule's equivalent of heat.

= 1400 ft. lbs. per degree Centigrade.

= 778 ft. lbs. per degree Fahrenheit.

E = Internal energy of 1 lb. of gas reckoned above 0°C or 32° F.

Let suffix 1 represent the initial condition of the gas and suffix 2 the final condition.

Boyle's Law. This Law states that if a gas is expanded or compressed at constant temperature, the product of its pressure and volume at any instant is constant, or

$$P \times V = \text{constant} = p_1 v_1 = P_2 V_2$$

Charles' Law :—If a gas is expanded or compressed at constant pressure, then $\frac{V}{T}$ = a constant.

If the change takes place at constant volume, then $\frac{P}{T}$ = a constant.

Characteristic Equation of gas :—By combining the laws of Boyle and Charles, the characteristic equation of a gas is obtained

$$Pv = RT$$

V = Volume of 1 lb. of gas at pressure P and absolute temperature T : R is known as the gas constant.

For atmospheric air, R = 96 ft. lb. Centigrade Units

= 53·3 ft. lbs. Fahrenheit Units.

(*b*) *Bulk Modulus of a fluid* :—The bulk elastic modulus of a fluid is the ratio between the increase of pressure and the volumetric strain caused by this pressure increase. This applies to liquids and gases.

Bulk modulus $K = -V\frac{dp}{dv}$: dp = increase of pressure

$-dv$ = change of volume

volumetric strain $= -\frac{dv}{v}$

The bulk modulus for a liquid is large on account of the small amount of compressibility. For a gas, the value of K is relatively small as dv is large.

(*c*) *Isothermal flow in pipes* :—This means that gas flows in a pipe without rise in temperature.

Consider a short length of the pipe dl at a point where the velocity is V, the pressure P, the density D, and the specific volume V. then

Weight of gas flowing $= Da\,v = D_1\,av_1$

$$p\,v = P_1 V_1$$

$$V^2 = \frac{P^2_1 V^2_1}{p^2}$$

$$\text{as } \frac{P}{D} = \frac{P_1}{D_1}$$

$$\text{Then } D = \frac{P D_1}{P_1}$$

$$\text{also } DV^2 = \frac{D_1 P_1 V_1^2}{P} : D_1 = \frac{P_1}{RT}$$

$$P_2 = P_1 \sqrt{\left(1 - \frac{8flv_1^2}{2gdRT}\right)} \quad \text{......} 113_1$$

P_1 = Pressure at the supply end of the pipe in lbs. per square inch.

P_2 = Pressure at the delivery end do do

L = length of the pipe in feet.

f = coefficient of friction.

T = absolute temperature.

V_1 = Velocity in feet per second at the supply end.

d = diameter of pipe in feet.

R = gas constant.

Example :—Compressed air is transmitted through 400 feet of 3″ pipe; the supply pressure is 150 lbs. per sq. in. and the flow is 180 cub. ft. per minute at the supply end. Calculate the delivery pressure assuming the temperature remains at 15°C throughout and that $Pv = 96\ T$ for 1 lb. of air.

Take $f = \cdot 005$

A = Area of 3 inch pipe = $\cdot 0490$ sq. feet.

Q = Flow = 180 cub. feet per minute = $3\cdot$ cu. ft. per second.

Velocity $= \frac{Q}{A} = 3 \div \cdot 049 = 61\cdot 4$ feet per second.

T = Absolute temperature = $15 + 273 = 288$

R = a constant = 96

By equation 113

$$P_2 = P_1 \sqrt{\left(1 - \frac{8flv^2}{2gd\ Tr}\right)}$$

$$= 150 \sqrt{\left(\frac{1 - 8 \times \cdot 005 \times 400 \times 61\cdot 22^2}{2 \times 32\cdot 2 \times \frac{1}{4} \times 96 \times 288}\right)}$$

$= 139\cdot 6$ lbs. per sq. inch.

For discharge of gas and atmospheric air through pipes, see Molesworth pocket book pages 649 and 650; Edition (1927).

127. *Buoyancy of a Balloon.*

According to the principle of Archimedes* which applies to bodies immersed in a gas in the same way as it applies to body immersed in liquids, the lift of the balloon is the weight of atmospheric air which it displaces minus the weight of the balloon and also minus the weight of the light air contained in the container attached to the balloon.

The lift would be maximum if there was vacuum in the container, but this condition is not possible as the container would collapse inwards due to the atmospheric pressure from outside. To prevent this, the container is filled with the lightest gas possible at a pressure nearly equal to that of the atmosphere.

The gas pumped in the container is generally Hydrogen or Helium.

Weight of air per cub. foot = $\cdot 0756$ lbs.

Weight of Hydrogen per cub. foot = $\cdot 0053$ lbs.

Weight of Helium per cub. foot = $\cdot 0116$ lbs.

For further information on the subject, see books on airships.

128. *Resistance of a sphere moving in fluid.*

The motion of a sphere moving through a fluid depends on the density and viscosity of the fluid and on the radius and velocity of the sphere.

If R be the resistance to motion then

R varies as $D^a u^b r^c v^d$

D = Density of the fluid

u = viscosity of the fluid

r = radius of sphere in feet

v = Velocity of sphere in feet.

$R = 6\pi r v u$ for a sphere with uniform velocity.

This is known as Stoke's Law. This law is used in the grading of fine powders, the particles of which are too small to be measured by direct means. Officers in charge of research laboratories, attached to the Irrigation Department for canals, use this law to find out the fineness of sand and silt in the canals.

A glass tube of about $1\frac{1}{2}$ inches diameter is graduated to some scale and fixed to a vertical board. The silt charged water from a canal or a river is put in the tube.

Time is noted as the silt deposits in the lower part of the tube from one mark to the next lower mark. Thus a comparative statement of the fineness of silt particles from various canals is made.

* Archimedes :—(287-212 B.C.) a Greek Geometrician and Philosopher of remarkable power. He discovered the principle of lever and specific gravity, also invented the famous Archimedian screw.

129. *Surface tension.*

When two liquids of different density are in contact, the surface of contact forms a curve called meniscus. The formation of curved surface is due to the attraction of the molecules and it is found that there is a slight pressure difference between the fluids on either side of the surface. The surface acts as an elastic skin which is in tension in both directions; this is surface tension.

This phenomenon is familiar in the bubble of a spirit level used by engineers.

In a large vessel, the surface of the liquid is curved near the perimeter only; the value of surface tension depends on the radius of the meniscus for a water surface in contact with the atmosphere, the surface tension has a value of about ·00042 lb. per inch at a temperature of 60° F.

130. *Two dimensional flow of a liquid.*

The motion of a liquid in a two dimensional plane may be in the form of a free cylindrical vortex, a free spiral vortex, a forced vortex and a radial flow.

(*a*) *A free cylindrical vortex* :—In this type of flow, the stream lines are moving freely in horizontal concentric circles and there is no variation of total energy 'E' across the stream lines. When the upper surface of the liquid is free, it assumes the parabolic shape. If a tube is fixed at the bottom of the cylinder, vertically, in the centre, to drain away the cylinder, then the vortex assumes the form of a free spiral vortex, as seen in a tornado, a water spout, etc.

(*b*) *A forced vortex* :—This is the case of stream of water moving in circles in the casing of a centrifugal pump, the whirl of the stream being caused by power from an external source. In a forced vortex, the liquid has a constant angular velocity.

$$V_a = \frac{V}{r}$$

V_a = angular velocity.
V = Velocity in a straight line, tangent to radius.
r = radius of vortex.

A forced vortex will have a centrifugal head impressed on the liquid, caused by its rotation.

$$\text{The centrifugal head} = \left(\frac{P_2 - P_1}{W}\right) = \left(\frac{V_2^2 - V_1^2}{2\,g}\right) \quad \ldots\ldots\ldots 114.$$

P_2 = intensity of pressure at the outer end of the outer radius
P_1 = do do do do inner radius
V_2 = Tangential velocity at radius R_2
V_1 = do do do R_1

(*c*) *Radial flow of a liquid* :—This has been dealt with in article 14·C Fig. 16 C. The liquid is flowing radially outwards between two horizontal flat discs, placed parallel and fixed distance apart. The liquid is supposed to enter at the centre of the upper or down disc through a hole by means of a pipe supplying water under a known pressure. The ends of the discs are open to Atmosphere. The water flows from the centre in streams along the radial lines. The path of the stream is a straight line, therefore the radius of the stream line is infinity, and change of Energy across the stream lines is 0. As the water flows between the plates radially outwards, the area of flow will increase and the velocity will decrease. This will cause an increase in pressure.

Velocity at any point at a distance of r from the centre is given by the equation.

$$V = \frac{Q}{2\pi rt} \quad \ldots\ldots\ldots 115.$$

Q = Discharge in cusecs
t = distance between discs in feet
r = radius of any point from the centre.

If the inner radius is r_1, and the outer radius is r_2 and V_1 and V_2 be velocities respectively, and also intensity of pressure be P_1 and P_2 at the ends of the radii r_1 and r_2, then $\frac{P_2 - P_1}{W} = \frac{V^2_1 - V^2_2}{2g} \quad \frac{Q^2}{8\pi^2 t^2 g}\left(\frac{1}{r_1^2} - \frac{1}{r_2^2}\right)$

If the values of P_1, P_2........ are calculated and plotted on a base representing r_1 and r_2........ the curve obtained will be a parabola, known as Barlow's curve. See article 14.

(*d*) *Free spiral vortex* :—This is a combination of radial flow case C and free cylindrical vortex case a. In a free spiral vortex the liquid is rotating and flowing radially at the same time, thus moving in the form of horizontal spiral, the resultant velocity of the fluid flows at a constant angle to the tangent at all radii.

Take the case of a metal cylinder 4 inches diameter, 3 inches high, closed entirely at the bottom, sides and top ; on the top, there is a hole $\frac{3}{4}$ inch diameter. On one side, there is a horizontal inlet at bottom $\frac{3}{4}$ inch diameter ; water under pressure enters this inlet, whose centre line is a tangent to the radius of the cylinder. Water churns round and round in the cylinder due to its entry from a pipe tangential to the cylinder. As the water enters the cylinder under pressure, it spouts from the hole in the top of the cylinder in the form of a spiral and sprays out. The height to which the water goes up depends upon the pressure available in the inlet pipe.

This appliance is called a sprayer and is used to spray out sewage-effluent on the top of a filter, to aerate it for purification purpose.

Such sprayers were put up by the writer in 1925 on the sewage disposal works at Kilokri (New Delhi) and at Kasouli (Simla Hills) for the new T. B. Sanatorium (Lady Linlithgow's noble work) in 1941.

131. *Determination of coefficient of viscosity.*

If a steel ball is allowed to fall through a column of the liquid, it will first accelerate, but the resistance to its motion increases with its velocity until it just balances the pull of gravity on the sphere. When the sphere reaches this stage, it falls with constant velocity, V, which is ascertained by timing the fall through a known height.

The resistance of the fluid according to Stoke's Law.

$R = 6 \pi u r V$ ....116.

R = resistance to viscous flow

r = radius of sphere

V = the constant velocity

If D = the denisty of the sphere and

D_1 = the density of the liquid.

Then pull of gravity on the sphere.

$= \frac{4}{3} \pi r^3 (D - D_1) g$ ....117.

This pull of gravity = fluid resistance on sphere at velocity V
or $\frac{4}{3} \pi r^3 (D - D_1) g = 6 \pi u r v$

from which $u = 2r^2 g \left(\frac{D - D_1}{9 V}\right)$ ....118.

Thus we get the coefficient of viscosity u.

It is presumed that the velocity V is low, no eddies are formed by the falling sphere, and the cylinder containing the liquid is of sufficient diameter to prevent its surface affecting the motion.

132. *Karachi sewerage scheme.*

The city of Karachi, built by the British on more or less flat ground, has been divided into several drainage areas, each being sewered separately and sewage collected at the lowest point into a sump wherefrom it is lifted by automatic Shone's Ejector, worked by compressed air, supplied from a central station. The compressed air forces the sewage out from the ejector's drum into a C. I. force-main which discharges into a gravitating outfall sewer, at the end of which the sewage is pumped for distribution on the sewago farm about 1,500 acres in area, without being subjected to any bio-aeration treatment.

In 1940, the writer was engaged to design sewerage scheme for a part of the Karachi cantonment area and had to propose a Shone's Ejector near Empress Market, the particulars of which are given below :

The ejector to consist of two drums or collecting chambers, each to store 200 gallons per minute : maximum quantity of sewage per minute 400 gallons on population basis : rising or force main of 9" C. I. Pipe, 1,860 feet long : Static lift for sewage, *i.e.*, the difference of level between the invert of rising main at discharge end and the level of the invert of the discharge pipe from the ejector drum is 17·36 feet :

Head lost in friction (See nomogram No. 3, Fig. 61A) through 9" C.I. Pipe, 1,860 feet long, for a discharge of 400 gallons per minute, including loss in bends = 11 feet, total Head = 17·36 + 11 feet = 29 feet. Air pressure in ejector $= 29 \times 62 \cdot 5 \times \frac{1}{144} = 12 \cdot 6$ lbs. say 14 lbs. per sqnare inch.

V_2 = Volume of free air required for one gallon of sewage : V_1 = Volume of one gallon of sewage = 0·16 c. ft. :
P_2 = Atmospheric pressure outside the ejector = 14·7 lbs. per square inch ;
P_1 = Pressure inside the ejector = 14 + 14·7 = 28·7 lbs. per sq. inch :

Boyle's Law $= P_1 \times V_1 = P_2 V_2$: or $28 \cdot 7 \times 0 \cdot 16 = 14 \cdot 7 \times V_2$: or $V_2 = 0 \cdot 31$ c. ft. Add 10 per centfor wastage, then $V_2 = 0 \cdot 331$ c. ft. : Thus free air required for 400 gallons of sewage = 400 × 0·331 = 132 c. ft. say 150 c. ft. Thus an air compressor of 150 c. ft. is required : Allow for one stand-bye, two compressors, each of 150 c. ft. are needed, or 3 of 100 c. ft. each.

133. *New Delhi water supply scheme.*

The following notes, collected by the writer during his service (1913-1927) in the P.W.D., New Delhi.

(*a*) *Pumping station at Wazirabad for raw water from the river Jumna*:—The raw water pumped here is of better quality than the water lower down in the river. Centrifugal pumps are used: The same are driven by high speed steam engines, supplied by boilers on the spot. Each pump is capable of lifting 350,000 gallons per hour against a total head of 26 feet, including 7 feet suction. The suction pipes have no foot valves, the suction vacuum being created by injecting steam into the suction pipes. Steam coal (2″ rubble) used in the boilers amounts to 4 cwts. per hour.

The raw water is pumped into a small preliminary settling tank and thence sent to main pumping station at Chandrawal, one mile lower down on the River Jumna:

(*b*) *Chandrawal pumping station*.—Here the raw water is admitted into large settling tanks old type, the latest pattern being at Dehra Dun Town and thence filtered through rapid filters (Pattersons) and pumped to a high level service reservoir for distribution to old city of Delhi and New Delhi. Messrs. Worthington and Simpsons supplied the steam pumps, worked by triple expansion steam engines (High Duty). Total head 120 feet: actual quantity of water pumped by each pump = 180,000 gallons per hour. Steam coal used 8 maunds per hour. Total quantity of water pumped 6 million gallons per day (1925): Patterson filters = 10 units; each unit to deal with 750,000 gallons per day of 24 hours: Alumino-ferric used in the coagulating tanks of filters = 1·3 grains per gallon. To sterilize the water, bleaching powder used per day = 108 lbs. After 1920, chlorine is used in place of bleaching powder: 2 lbs. of liquid chlorine are used for each million gallons of filtered water. No ill-effects due to chlorine. To cope with the increased demand for water-supply, centrifugal pumps worked by electric motors, have been installed lately.

134. *Long lines of large pipes*:—Often long lines of pipes are laid to carry water to far-off places. The difference of even one inch diameter in the size of the pipe would make an appreciable difference in cost: Therefore the diameters should be calculated on modern formula, based on modern experiments conducted by specialists. In formula 52A, the value of μ, (often written as *f*) is given below for different pipes of different kinds. (See Journal of the I.C.E., 1937-1938 and Proc. Roy. Soc. Vol. 161).

TABLE No. 28.

Values of Coefficient *f* for Pipe Flow Formula.

$$hf = \frac{4f + LV^2}{2gd}$$

Diameter in inches.	Smooth Pipes.			Clean Asphalted Pipes.			New Cast Iron Pipes.			Old C. I. Pipes.		
	Velocity in feet per second.											
	1–3	4–6	8–20	1–3	4–6	6–8	1–3	4–6	10–15	1–3	4–6	10–15
3	·0066	·0047	·0042	..	..	..	·0067	·0063	·0060	·0152	·0135	·0122
	·0050	·0043	·0043	..	..	..	·0064	·0061	·0059	·0139	·0128	·0120
6	·0055	·0042	·0035	·0102	·0086	..	·0061	·0057	·0054	·0135	·0114	·0103
	·0043	·0038	·0030	·0090	·0081	·0079	·0058	·0055	·0053	·0117	·0108	·0100
9	..	..	..	·0080	·0067	..	·0057	·0053	·0050	·0122	·0110	·0092
	..	..	..	·0070	·0064	·0060	·0054	·0052	·0049	·0115	·0100	·0090
12	·0047	·0035	·0032	·0067	·0056	..	·0054	·0050	·0048	·0108	·0094	·0084
	·0038	·0033	·0027	·0059	·0053	·0050	·0051	·0049	·0047	·0096	·0089	·0080
18	·0044	·0033	·0029	·0056	·0046	..	·0052	·0048	·0041	·0087	·0076	·0069
	·0035	·0030	·0025	·0048	·0043	·0042	·0050	·0045	·0040	·0078	·0073	·0067
24	·0042	·0032	·0028	·0050	·0042	..	·0050	·0043	·0037	·0076	·0065	·0060
	·0033	·0029	·0024	·0044	·0039	·0038	·0045	·0040	·0036	·0067	·0063	·0059
30	..	..	..	·0047	·0039	..	..	..	..	..	..	..
	..	..	..	·0041	·0037	·0036	..	..	..	..	..	..
36	·0037	·0029	·0026	·0044	·0036	..	·0042	·0036	·0037	·0061	·0054	·0050
	·0031	·0027	·0023	·0038	·0035	·0034	·0037	·0035	·0036	·0056	·0052	·0049
48	·0035	·0028	·0025	·0039	·0033	..	·0036	·0032	·0029	·0057	·0050	·0046
	·0029	·0026	..	·0035	·0031	·0029	·0032	·0031	·0028	·0051	·0049	·0045
60	·0034	·0027	·0025	..	..	..	·0032	·0030	·0028	..	..	..
	·0028	·0025	..	..	..	..	·0030	·0029	·0027	..	..	..

The value of *f* for a given pipe, given velocity, given discharge can also be ascertained experimentally.

INDEX

INDEX—*Contd.*

INDEX—*Contd.*

NOMOGRAM II : F I (S VOL CXXII. 1894). OR KUTTER'S FORMULA
WITH 'N' BETWIN .012 AND.